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DISCUSSIONS

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* Publication of closing discussion pending.

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AMERICAN SOCIETY OF CIVIL ENGINEERS
Founded November 5, 1852
PAPERS

FOUNDATION EXPERIENCES, TENNESSEE VALLEY AUTHORITY

A SYMPOSIUM

NOTE.—Written comments are invited for immediate publication; to insure publication the last discussion should be submitted by **July 15, 1940.**

FOREWORD

When the Tennessee Valley Authority was established by Congress, in 1933, its zone of influence included two completed power developments—Wilson Dam, a federal project, and Hales Bar Dam, a private project. The new authorization extended the development of the valley by the proposed construction of additional dams. An important by-product of this activity—especially to the civil engineering profession—has been the imposing mass of technical data thus made available.

The papers of this Symposium, restricted as they are to foundation experiences, demonstrate the comprehensive nature of the entire project. Three dams are selected as being typical for this purpose—Norris, Guntersville, and Chickamauga. In each paper the author has confined his scope either to geology or to the practical foundation treatment required at the dam site under consideration.

In an effort to avoid the appearance of advocating special commercial interests, the names of manufacturers of equipment described in this Symposium have been omitted. To the same end, it is expected that discussers will confine their comments to matter within the intended scope of each paper.

Personnel.—The projects were built with the Authority's forces. At the beginning of this work C. A. Bock, M. Am. Soc. C. E. (chief consulting engineer) was assistant chief engineer; T. B. Parker, M. Am. Soc. C. E. (chief engineer) was chief construction engineer, later to be succeeded by A. L. Pauls, M. Am. Soc. C. E.; Edwin C. Eckel, Affiliate, Am. Soc. C. E., was chief geologist; and Berlen C. Moneymaker, assistant chief geologist.

On the Norris Dam, Barton M. Jones, M. Am. Soc. C. E., was project construction engineer; C. Douglas Riddle, Assoc. M. Am. Soc. C. E., assistant construction engineer, was succeeded successively by Frederick A. Dale and A. M. Komora, Members, Am. Soc. C. E.; and Ross White, M. Am. Soc. C. E., superintendent of construction, was succeeded by F. C. Schlemmer, who subsequently became construction superintendent of Chickamauga Dam.

At Chickamauga Dam, Lee G. Warren, M. Am. Soc. C. E., was project engineer; James B. Hays, M. Am. Soc. C. E., construction engineer; J. K. Black, Assoc. M. Am. Soc. C. E., assistant construction engineer; and G. E. Murphy, assistant construction superintendent, succeeded by James S. Lewis, Jr., Assoc. M. Am. Soc. C. E.

At Guntersville Dam, Verne Gongwer, M. Am. Soc. C. E., was project engineer; George K. Leonard, M. Am. Soc. C. E., construction engineer, succeeded by F. E. Bell; H. L. Broadfoot, assistant construction engineer; George P. Jessup, construction superintendent, succeeded by A. W. Sherman; and B. S. Philbrick, M. Am. Soc. C. E., assistant construction superintendent.

FOUNDATION TREATMENT AND RESERVOIR RIM TIGHTENING AT NORRIS DAM

By James S. Lewis, Jr.,[1] Assoc. M. Am. Soc. C. E.

Synopsis

The foundation of Norris Dam, in Tennessee, is composed of dolomite, a stratified rock of sedimentary origin. Preliminary investigations disclosed that the formation was characterized by the existence of openings that would permit the passage of water under the structure. The certainty that damaging leaks of large volume would result when the reservoir was filled, if the openings in the foundation were not sealed, made it desirable that a comprehensive program of foundation treatment be included in the plans for the construction of the dam. It was also necessary to treat parts of the reservoir rim.

Under the dam, the treatment was divided into two parts: Shallow, low-pressure grouting of the foundation area to reduce uplift and to consolidate the rock; and deep, high-pressure grouting to form an impermeable curtain under the heel of the structure. On the reservoir rim, the work involved the determination of the location of the parts in need of treatment and the grouting of the parts found to be faulty.

Under the structure, all seams intercepted by the holes for both shallow and deep grouting were carefully washed free of unsound material before grouting. The holes for the shallow grouting were divided into interlocking patterns, whereas those on the line of the curtain were drilled in groups. The effectiveness of the work was improved greatly by the series of operations made necessary by this arrangement.

In the reservoir rim, consolidation rather than replacement of the material in the seams was desired, and washing was not attempted. It was realized that a large volume of material would be required for this work and, in the interest of economy, rock flour was added to the cement grout.

This paper contains a description of the equipment used and of the field practices found to give best results in handling the problems that arose. Cost and progress records are included.

In an effort to avoid the appearance of advocating special commercial interests, the names of manufacturers of the equipment described in this paper have been omitted. To the same end, it is expected that discussers will confine their comments to matter within the intended scope of the paper.

[1] Asst. Constr. Supt., Watts Bar Dam, TVA, Spring City, Tenn.

Introduction

The construction of Norris Dam by the Tennessee Valley Authority (TVA) was begun in October, 1933, and the project was officially dedicated on March 4, 1936. The maximum height of the dam is 265 ft and the length 1,860 ft. Fig. 1 is a photograph of the completed structure.

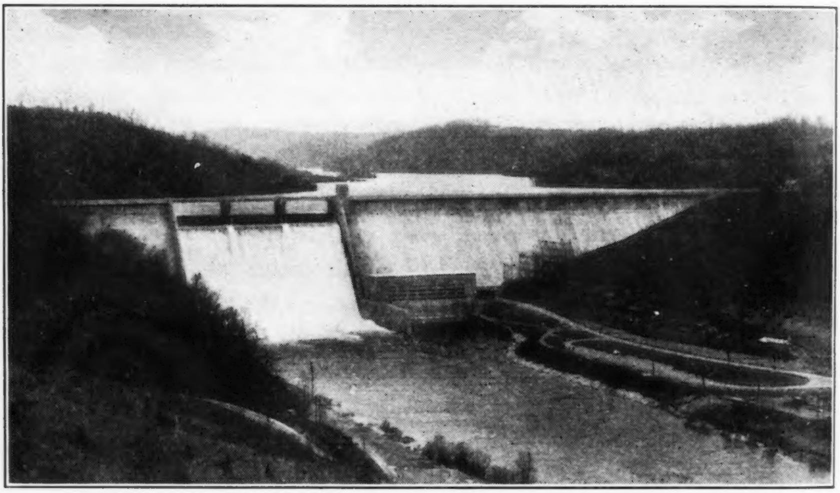


Fig. 1.—Norris Dam

The history of the treatment of dam foundations has proved the value of grouting before water is impounded if the nature of the foundation is such as to indicate that excessive leakage may occur. It is expensive and discouraging to attempt to stop foundation leaks that appear after a reservoir has filled, and the TVA determined to spare no reasonable efforts in its attempt to prevent difficulties of this nature at Norris Dam. The foundation of the structure and those parts of the reservoir rim near the abutments were investigated by means of diamond core holes, drilled as part of the preliminary studies made in determining the location of the site. The information obtained in this manner was later supplemented by a geological and a geophysical survey of the site and the region. From the knowledge secured, it was ascertained that the filling of the seams and other openings that existed in the rock would be an undertaking of major proportions.

The exceptional care that was exercised to prevent the occurrence of leaks through the foundation and the reservoir rim of the dam has well repaid the effort that was expended. The volume of both drilling and grouting was large, and it was necessary to use a variety of methods to cope with the problems that arose during the prosecution of the work.

Geology

The geological survey revealed that the rock on which the dam would rest is composed of a hard dolomite banded with numerous seams. Before grouting

operations started some of the seams were filled with clay and some were partly open. In addition, the formation is characterized by the existence of large solution channels, weathered joints, and fissures. At the site, the dip is about 5° in a downstream direction, the strike being roughly parallel to the axis of the dam. The rock in the immediate vicinity of the chosen location is known as Rockhouse dolomite, a subdivision of the Knox dolomite that underlies the region. The name Rockhouse evidently alludes to the numerous caves which are found along the bluffs and cliffs bordering the river and which could afford shelter in case of necessity. This brief description of the geology of the region is intended to afford a picture that is typical of the dam foundation and of the reservoir rim. Numerous extensive seams, caverns, and solution channels would have to be sealed in order to insure against serious leakage.

Early Foundation Investigations

Closure of the first cofferdam required some grouting in order to stop leaks through seams into the enclosure. In this preliminary work, it was learned that both open and clay-filled seams were so numerous and extensive that only a systematic plan of treatment would succeed in tightening the foundation effectively. Accordingly, after some study, a plan was evolved whereby the entire foundation was to be drilled on a grid of wagon and core-drill holes at

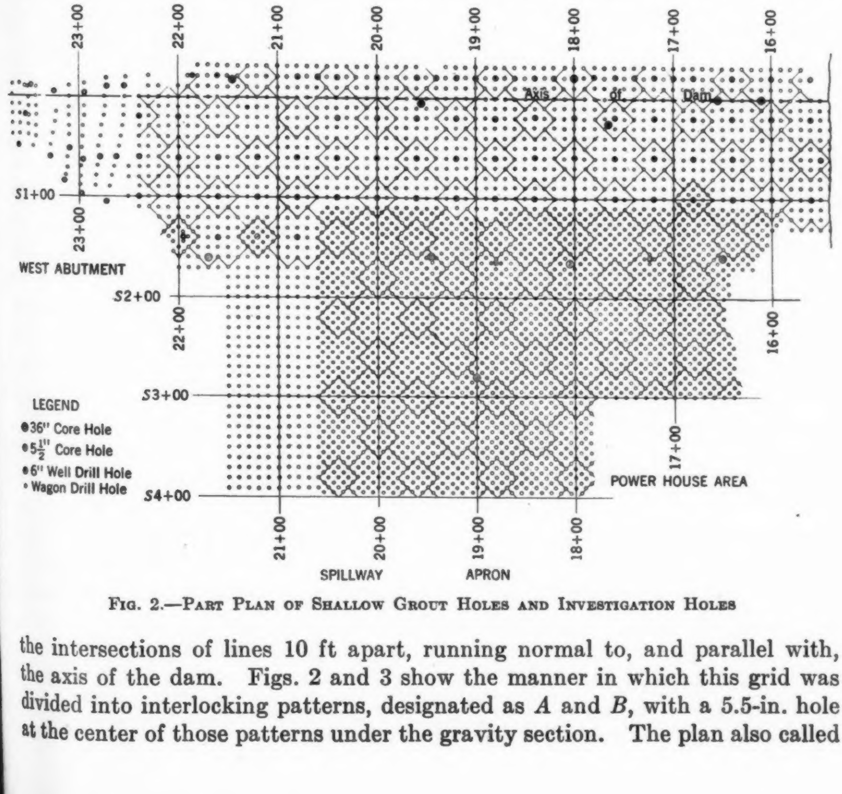


Fig. 2.—Part Plan of Shallow Grout Holes and Investigation Holes

the intersections of lines 10 ft apart, running normal to, and parallel with, the axis of the dam. Figs. 2 and 3 show the manner in which this grid was divided into interlocking patterns, designated as *A* and *B*, with a 5.5-in. hole at the center of those patterns under the gravity section. The plan also called

for the exploration of the holes in order to locate all seams, and the thorough washing of these seams to remove clay and loose material. As soon as the foundation rock was exposed in the first cofferdam, the drilling of exploratory holes was started with wagon drills and shot core drills.

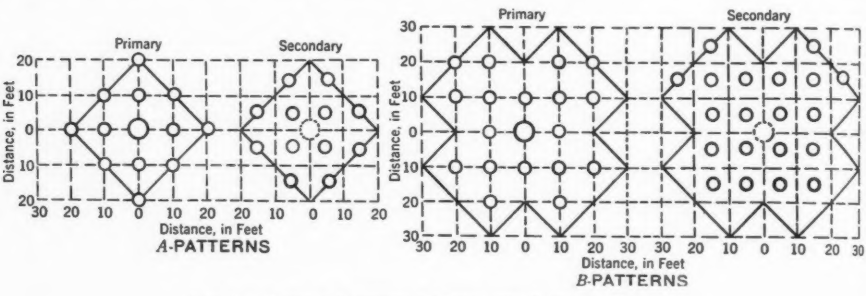


FIG. 3.—TYPICAL PATTERN LAYOUT AND DRILLING DIAGRAM

Locating Seams with Wagon Drills.—It was hoped that the operators of the wagon drills could determine the occurrence and thickness of seams, satisfactorily, and report them to a recorder. However, it was sometimes difficult to distinguish between an opening and a soft place in the rock. Drill operators could not be relied upon to maintain the constant vigilance necessary to obtain such information as might have been available. Reliable information could not be obtained as to the size of seams because of the tendency of the rock to spall from the top of a seam and pile up under the bit as the drill penetrated it, thus making it difficult to determine the actual drop. In general, it can probably be said safely that wagon drills do not afford a reliable means of locating seams less than 3 in. in thickness. However, although much less effective for exploration purposes than core drills, the relative economy of wagon drills justifies their use under many conditions. The cost of $5\frac{1}{2}$-in. shot core holes was \$3.66 per ft as compared with \$0.408 per ft for wagon drills.

Exploration of Drill Holes.—In order to reduce the uncertainty attending the information obtained from the drill operators, an exploring instrument known on the job as a "Feeler" was devised for the purpose of locating seams and measuring their thicknesses. This device (see Fig. 4) consisted of a pair of steel legs so hinged that the weight of the instrument caused them to bear outwardly against the wall of the hole. When a seam was encountered, and the wall of the hole no longer acted to restrain the legs, they snapped vigorously outward with a force that was easily felt by one handling the explorer on a line from the surface. The thickness of the seam was measured by the free vertical movement that was permitted. The presence of clay in a seam caused no difficulty, as such material is always dislodged for some distance back from the wall of the hole. The device was suspended on a marked $\frac{1}{4}$-in. wire rope and was equipped with a latch that released the legs automatically when the bottom of the hole was reached. The hole was explored by pulling the "Feeler" from the bottom upward. The method was found to be satisfactory in wagon-drill holes and, by increasing the length of the legs, it was used with even better

results in vertical $5\frac{1}{2}$-in. core holes. It is not desired to imply that the method has no shortcomings. Its effectiveness depends to a considerable extent upon the experience of the operator and his skill in interpreting the indications that he obtains. In rough-walled holes, such as those drilled with wagon and churn drills, the effectiveness is not so great as in the comparatively smooth core holes.

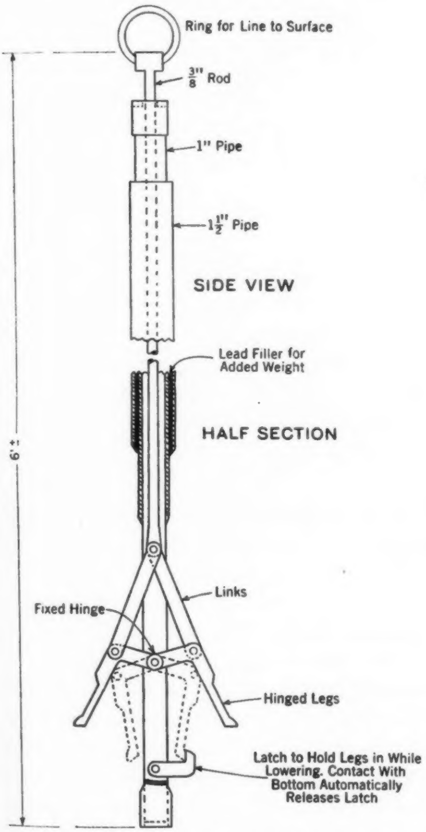


FIG. 4.—MECHANICAL HOLE EXPLORER

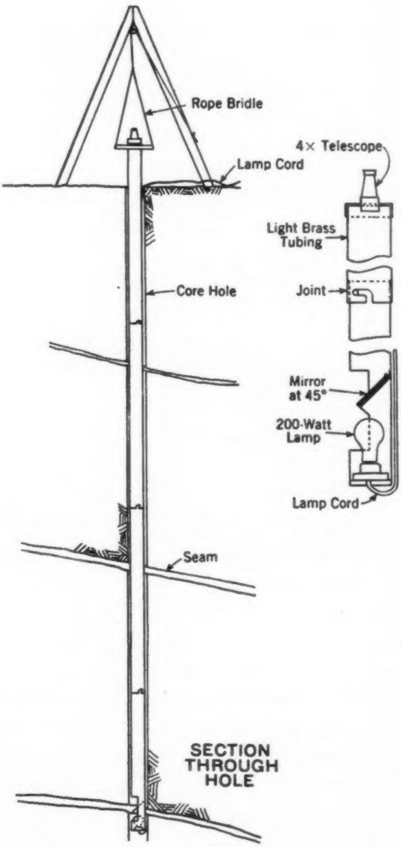


FIG. 5.—PERISCOPE FOR INSPECTING CORE HOLES

Periscope.—A periscope (see Fig. 5) was designed for checking the results of the exploration of the $5\frac{1}{2}$-in. core holes by the "Feeler." The periscope was made of brass tubing in sections 10 ft long, graduated in feet to facilitate reading depths. It was used at a maximum depth of 75 ft; but, in the form described, it was not suitable for use under water or in crooked holes. If the length of section was reduced to 5 ft it could be handled more easily by the observer as, with long sections, he must either be lowered in a sling or must climb down a ladder while keeping his eye at the telescope. This periscope

was used successfully to test the effectiveness of the explorer, after which it was abandoned in favor of the simpler and faster "Feeler."

Foundation Studies.—During the early stages of the excavation, it was considered desirable to secure as much information as possible so that it could be used in formulating a plan to be applied flexibly to the foundation treatment of the entire project. For this purpose five $5\frac{1}{2}$-in. shot drills and one 36-in. shot drill were distributed over the exposed area of the foundation, and exploration holes were drilled to depths varying from 40 to 80 ft. Using the information obtained from the holes, sections parallel with, and normal to, the axis of the dam were plotted to show the location of the seams. On these sections, a continuity of gently rolling seams, dipping slightly downstream, could be discerned, as well as a similarly rolling, almost horizontal, stratification parallel with the axis of the dam. The seams were extensive in area and varied in thickness from mere contact lines of bedding planes to openings of several inches. Some were water bearing whereas others were clay-filled or open. As evidenced by the cores, the rock was an excellent quality of hard, close-grained dolomite that tested 25,000 lb per sq in. in compression. At a depth of 28 ft below the surface, a layer of badly broken, seamy rock was found to exist generally under the entire area of the foundation. The thickness of this fractured band was approximately 3 ft, but the rock was of such quality that no apprehension was felt as to its ability to carry the load imposed by the structure. Since it was felt that this condition could be corrected satisfactorily by grouting, it was decided that it would be unnecessary to excavate the material.

Large Core Holes.—The true value of the 36-in. drill was made apparent in this early stage of the exploration as, at best, the knowledge that may be obtained from small core holes is limited. Grinding and blocking of the core within the core barrel act to destroy the original appearance of the rock and it is often difficult to judge whether core loss results from attrition or from a seam. This fault may be reduced to some extent by forcing the drillers to pull the cores frequently. Actually, it is the operator of the drill who must interpret, by the action of his machine, the evidence as to subsurface conditions signaled through the tools. There is no wish to detract from the value of borings of small size but rather to emphasize the value of large borings. The value of an investigation made by means of small holes is greatly increased when it is supplemented by holes large enough to permit the entry of a man. In the large holes, the rock may be studied in its original, undisturbed state to an extent that is not possible in shafts that have been excavated with explosives, wedges, and percussion drills. The large cores do not suffer from grinding and blocking as they do not rotate within the drill bit.

The greatest limitation upon this work is imposed by the ground water, and the depth at which the inflow becomes so great that the driller is unable to work in the bottom of the hole to remove core usually fixes the depth to which the hole may be drilled without sealing the rock in advance. A crew consisted of one driller with two helpers, and the average drilling rate was 0.33 ft per hr.

Holes reaching a depth of 57 ft were drilled for exploration purposes by first drilling four wagon-drill holes on the corners of a 10-ft square and grouting these holes before the core hole was started in the center of the square. Considerably greater depth could be reached in any case by advance grouting of the region to be penetrated.

FIG. 6.—TYPICAL FOUNDATION

In determining the general plan of treatment to be followed, the knowledge obtained from drill records, exploration records, inspection of 36-in. holes, and observations of the structure of exposed rock was correlated to form the basis of the final plan. As the result of this study, it was decided to remove the rock under that part of the gravity section lying in the river bed down to a clay-filled seam that had been found to underlie the rock surface at a depth of approximately 12 ft. Under the spillway apron and power house the excavation was to be extended only sufficiently deep to meet construction requirements or to remove weathered and unsound material. The surface of all of the rock exposed at seams or bedding planes was found to be dipping gently downstream and was marked by frequent domes and hollows that would afford an excellent mechanical bond to resist sliding. The great frequency of these irregularities is shown clearly in Fig. 6.

Drilling.—Referring to Figs. 2 and 3 showing the layout of the holes and the system of interlocking patterns that was chosen for the shallow grouting, the plan called for the drilling, washing, and grouting of the *A*-patterns in an area before the *B*-pattern holes were drilled. Areas were ordinarily not less than 100 ft square. The grouping greatly facilitated the washing of unsound

material from the seams and the later replacement of this material with grout, as the manner in which the *B*-patterns surrounded the *A*-patterns permitted a cross flow between holes that covered practically every square foot of the area. Grouping also allowed a large number of holes to be drilled simultaneously, thus contributing toward the improved progress of construction at a stage when delays could prove very costly.

Washing Seams for Shallow Grouting

The preliminary grouting to seal the first cofferdam revealed that the holes in any group were usually interconnected by seams and that it was frequently possible to wash away large quantities of clay and unsound material by forcing water and air into different holes of the group. The water emerging from the other holes would bring part of the material to the surface and it is probable that large quantities of loose material were also washed to distant areas in the seams. A mixture of air and water was found to be more effective than water alone, as the expansion of the air produced a turbulent action that increased the erosive powers of the fluid. The air also acted as a lift to bring the material to the surface through other holes.

Flow-Reversing Device.—To improve the dislodging effect of the washing fluid, a small air receiver with a reducing valve on the inlet and a four-way valve on the discharge, as shown in Fig. 7, was used for reversing the flow of

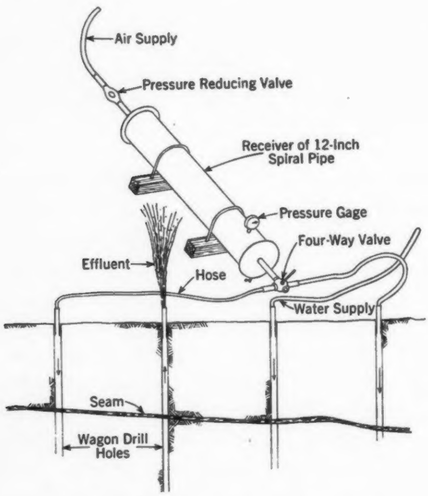


Fig. 7.—Seam Washing Arrangement

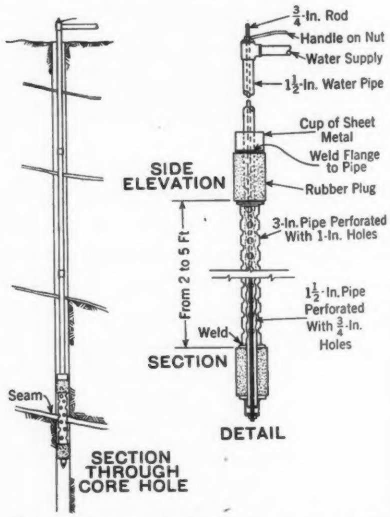


Fig. 8.—Compression-Type Double Expander for Washing Seams

the air. In washing a pattern, it was customary to connect a supply of water to one hole and to connect air from the receiver to holes on opposite sides of the water connection. By operating the four-way valve manually, at frequent intervals, the water in the seams was maintained in a turbulent state that was

very effective in dislodging clay and decomposed rock. An air motor mounted on the receiver to operate the four-way valve at regular intervals would improve the efficiency of this equipment. When the washing operation was begun, most of the holes were capped and, as the flow from the open holes became clear, the caps were moved about to change the path of flow of the fluid. In this manner each pattern was washed until the clarification of the effluent water indicated a reasonable freedom from clay and other undesirable material. The methods described were used under the spillway apron and power house.

Washing Individual Seams.—Under the gravity section, where it was desirable to secure an exceptionally tight seal, each seam was isolated and washed individually by means of a double expander (see Fig. 8) which was inserted in the $5\frac{1}{2}$-in. core hole at the center of the pattern after exploration of the hole had disclosed the location of the seams. The expander was placed so that the rubbers were an equal distance on each side of the lowest seam. The expanding nut was then tightened and water and air forced in until all possible interconnection with the wagon-drill holes in the group had been established. As soon as reasonably clear water flowed from a hole, it was capped in order to force a passage to other holes. Frequently, holes that offered great resistance at the beginning of washing operations would open and take water quite freely after pressure had been applied for several hours. The washing was always started at the lowest seam in a hole, and progressed upward. Normally, the rubbers on the double expander were spaced 24 in. apart, but, when a closely seamed zone that exceeded this spacing in thickness was encountered, the spacing was increased. At the same time that the seams underlying a pattern were being washed individually through the core hole, the air receiver used for producing turbulent flow was connected to the wagon-drill holes. At intervals while washing, and to complete the washing operation, every hole was blown clear of accumulated muck with an air jet. The washing was usually timed so as just to precede the grouting. The length of time required for washing a pattern varied between 6 and 24 hr and was largely determined by the judgment of the inspector. Ordinarily, when washing was started, the overflow water from interconnected holes was very muddy, containing small lumps of clay and weathered rock. After some time, the water gradually would become clear, although it was considered neither practical nor desirable to wash until the discoloration had disappeared entirely.

Pressures of air and water for washing shallow grout holes were limited to 30 lb per sq in. and it was necessary at times to reduce this pressure in order to avoid lifting the rock. The safe pressures for washing and grouting were determined by learning the depth to the uppermost seam. The weight of the rock over this seam was then computed. Ordinarily, the theoretical pressure could be exceeded as it was safe to assume that pressure would not exist under the entire area and that the surrounding rock would offer restraint through slab action.

Upheaval Gages.—Upheaval gages, to give an indication when the safe pressure was being exceeded, were placed at intervals over the area of the foundation. These gages (described subsequently) were observed during both

washing and grouting operations and frequently served as a warning that the safe pressure was being exceeded.

Shallow Grouting

In general, the system that has been described was used in preparing for the shallow grouting of the foundation area, but it became desirable at times to modify these methods to adapt them to special conditions. Under the spillway apron and power house, where a number of small seams were known to exist near the surface, the *A*- and *B*-patterns were first drilled and grouted to a depth of 20 ft. Then a secondary system of 40-ft holes, evenly spaced between those first drilled, was superimposed upon the original patterns and the same sequence of operations was repeated. In washing and grouting the 40-ft holes, it was not unusual to find a slight connection with nearby 20-ft holes that had been previously grouted to refusal. It was assumed that subsequent shrinkage of the grout deposited in the first operation had opened the seams slightly. Occasionally, throughout the duration of the work, it was found that open passages existed in territory that had been grouted. Besides shrinkage of the grout in the seams, this was attributed to the fact that, when washing in an area which had been previously tightened by grouting, it was possible to build up higher pressures because of the more limited and restricted passageways that existed. The resulting velocity of the wash water was probably considerably higher than in the first operation, and it was safe to assume that unsound material which had previously resisted the lower velocities of the first washing was scoured out. New passages were thus opened. To a lesser extent, this same progressively restricting effect was utilized by adopting the pattern system. The grouting of the *A*-patterns effected some consolidation of the ungrouted *B*-pattern areas, and the greater resistance to flow that resulted when the *B*-patterns were washed increased the velocity and improved the scouring effect.

The pattern system of grouting was followed, in general, over most of the foundation area. Only under the spillway apron and in a small area in the east abutment was the primary pattern system followed by a secondary system extending to a greater depth. Under the gravity section, grouting was always started from the core hole at the center of the pattern and the wagon-drill holes were capped as overflow occurred. After refusal had occurred on the core hole, the wagon-drill holes were grouted individually and ordinarily required very little grout. In general, the area under the spillway apron and power house was grouted to a depth of 40 ft, whereas the area under the gravity section, where the excavation was extended 12 ft deeper, was grouted to a depth of 30 ft.

In order to meet special conditions more adequately or more economically, it was necessary, occasionally, to vary the plans outlined herein. In a relatively small area under the west side of the spillway apron, the plan was altered to expedite the work during an important stage of construction. The alteration simply consisted in dividing the area into three parts that were drilled and

grouted separately, the holes being placed at the intersections of lines forming a 10-ft grid.

The west abutment was composed of a number of narrow ledges underlain by seams, and it appeared desirable to discontinue the pattern system and to treat each seam separately. In the east abutment (not shown in Fig. 2), the ledges were much wider and the pattern system was modified to obtain the best results on each one.

Anchorage of Spillway Apron.—In each of the wagon-drill holes in the area of the spillway apron, a bar of $1\frac{1}{4}$-in. steel was inserted to anchor the concrete apron slab against uplift. When early and wide interconnection developed in a pattern, filling other holes with grout that might harden and prevent the insertion of the rods, the difficulty was overcome by placing tees with plug cocks on the side outlets on all nipples. When grout flowed from a connected

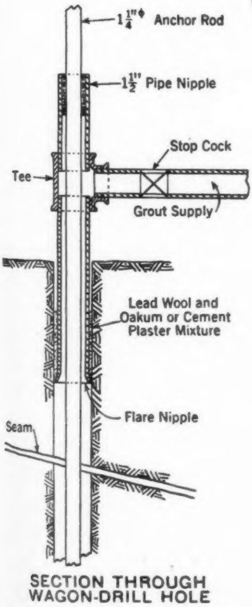


Fig. 9.—Arrangement for Grouting Seams and Anchor Rod Simultaneously

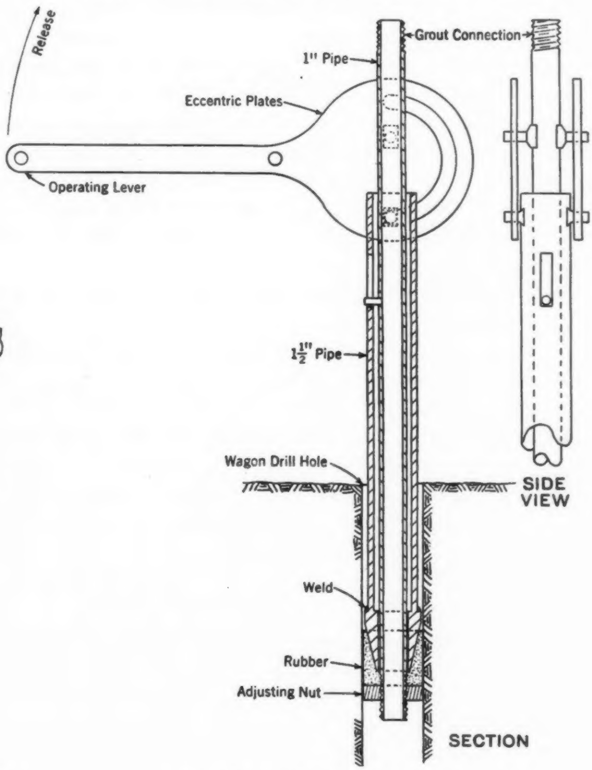


Fig. 10.—Expander for Small Drill Holes

hole, the rod was inserted through the tee as soon as the density of the overflowing fluid appeared to equal that of the mixture being injected. The cock on the side outlet was opened until oakum could be calked around the rod at

the top of the tee, after which it was closed to permit pressure to build up. Fig. 9 shows the details of the arrangement. When refusal on the original hole had occurred, each hole in the group was grouted through the side outlet of the tee.

Surface Leaks.—When the washing indicated that leakage of grout could be expected to occur where seams intersected the surface, these seams were calked in advance with oakum and lead wool, driven in with a blunt-edged calking tool and backed up with wooden wedges or a dry mortar of quick-setting cement. Where a seam appeared at the base of a vertical face, a rough form was sometimes built a short distance from the face and filled with concrete through which pipes discharged any water that might be flowing from the seam. The pipes were later capped when grout flowed from them. When it was necessary to calk a seam from which grout was flowing, lead wool and oakum were found to be most effective. Jackhammer holes were first drilled into, and along, the seam, and 1-in. pipe nipples were calked in so that they might serve as drains while the seam was being plugged. When the seam had been successfully calked (often a difficult and discouraging task), the pipes were capped. The flow of grout was maintained at all times during the calking, despite some apparent waste of material, as it was only in this manner that the loss of holes could be prevented. The value of grouting, properly performed, is inestimable, and it is impossible to avoid some waste in maintaining a proper standard for the work. Holes made useless (usually termed "lost"), as the result of a temporary cessation of grout flow, must be replaced by additional holes and even this may not insure that the area will be grouted satisfactorily.

Frequently it was found that the feather edge resulting where a seam intersected the surface of the rock at a small angle caused trouble by lifting when the seam was grouted under pressure and that calking of the leak only resulted in additional uplift. In such cases, it was usually necessary to reduce the grouting pressure and to pump slowly with a thick mix, allowing some grout to waste until the seam had filled and plugged. When it was learned in advance that such a condition existed, jackhammer holes were drilled vertically from the surface, just deep enough to penetrate the seam, and it was grouted at a low pressure of 5 or 10 lb per sq in. The deep holes for the patterns were then drilled through this seam and grouted as usual.

Hole Connections.—As soon as the drilling of a pattern had been completed, the holes were blown clear of muck and a $1\frac{1}{2}$-in. nipple, 18 in. long, was placed in the top of each. The nipples were preferably grouted in with a quick-setting cement mixture but were calked in with lead wool if time was pressing. A mixture consisting of equal parts of portland cement and gaging plaster was found to set very quickly and to have sufficient strength to hold the pipe nipples firmly. A quick-working, simple expander was developed after the shallow grouting was completed, and it is certain that economy could have been realized from the use of this device to replace the nipples. The expander is shown in Fig. 10 and its effectiveness has since been tested in the field at Chickamauga Dam near Chattanooga, Tenn.

Equipment.—For mixing the grout, two-compartment, open-top, mechanically agitated mixers that discharged by gravity into the suction of a 7 by 5 by 10, air-driven, duplex, reciprocating pump, were found to be very satisfactory. A corner-type, air-driven drill motor, mounted on a bracket at the end of the mixing tank, was used to drive the agitating blades. The pump and mixer were mounted together as a portable unit that was easily moved to any part of the job. The mixer was divided into two compartments so that an uninterrupted flow to the pump might be obtained by mixing in one compartment while discharging the other. A valve in the branch of the pump suction from each compartment permitted the pump operator to maintain a continuous flow. Fig. 11 shows the arrangement of the equipment in a unit.

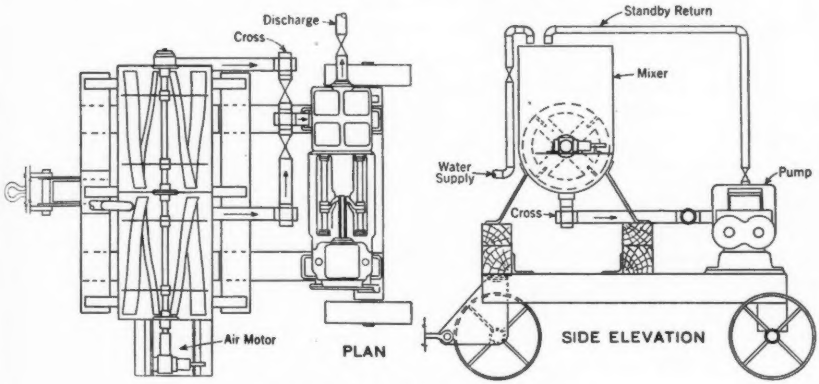


FIG. 11.—PORTABLE GROUT MIXER

The maintenance of the mixers was not an important item, as wear of the shaft and the stuffing boxes where the shaft passed through the end walls and partition was not excessive if reasonable care was exercised in keeping the glands packed. On the pumps, the demand for the repair and replacement of valves, piston packing, and cylinder liners was more or less constant. The fluid pistons were of cast iron with rubber packing, and the cylinders contained removable steel liners. The rate of wear of the rubber piston packing against the steel liners was found to depend largely upon the pressure at which the pump was operating and the wear was approximately equal on the steel and on the rubber. The life of the cast-iron valve seats was also found to depend to a great extent upon the pumping pressure. The seat rings were removed in the field and refaced in the shop. Valve disks of medium rubber were found to be far superior to the fiber disks furnished as original equipment because their greater ability to conform to slight irregularities in the seat made them more effective in preventing leakage. The abrasive action of grout is so great that, once a small leak has started, only a short time is required for considerable damage to result. Piston packing and valves were usually replaced in the field, although it was necessary to have cylinder liners removed and replaced in the machine shop. Four complete grouting units were kept in service, and

one spare fluid end for the pumps was always held in good repair to facilitate replacements.

The greatest source of trouble with regard to wear on the pumps was found to lie in the steel particles that were contained in the cement. These particles blocked valves and scored cylinders and, in general, increased the wear on the pumps appreciably. At various times during the grouting operations, screens were installed to remove foreign material from the fluid grout, but they proved such a constant source of trouble and delay that it became desirable to abandon their use. Dry screening of the cement through a screen small enough to remove the particles would have been an expensive operation, and subsequent experience indicated that an agitated sump, into which the foreign material could settle, between mixer and pump, probably would offer the best solution to the problem. A mixer that includes such a sump was developed by James B. Hays, M. Am. Soc. C. E., for use by the Bureau of Reclamation.

Grouting Procedure.—It was always considered desirable to place the mixer as near the hole being grouted as practicable; but construction activities and the necessity of hauling cement to the location usually governed the choice. Frequently, it was necessary to pump the grout for a distance of several hundred feet and a modified circulating system was chosen as offering the most satisfactory and economical method of handling it. As may be seen in Fig. 12, the grout was pumped to a header connected to the nipple at the hole, where the flow and pressure could be controlled with valves so that the desired pressure was maintained at all times. This control is necessary when the refusal pressure is determined in advance, as the characteristics of a hole may be such that, at the rate of pumping maintained, the refusal pressure occurs as soon as

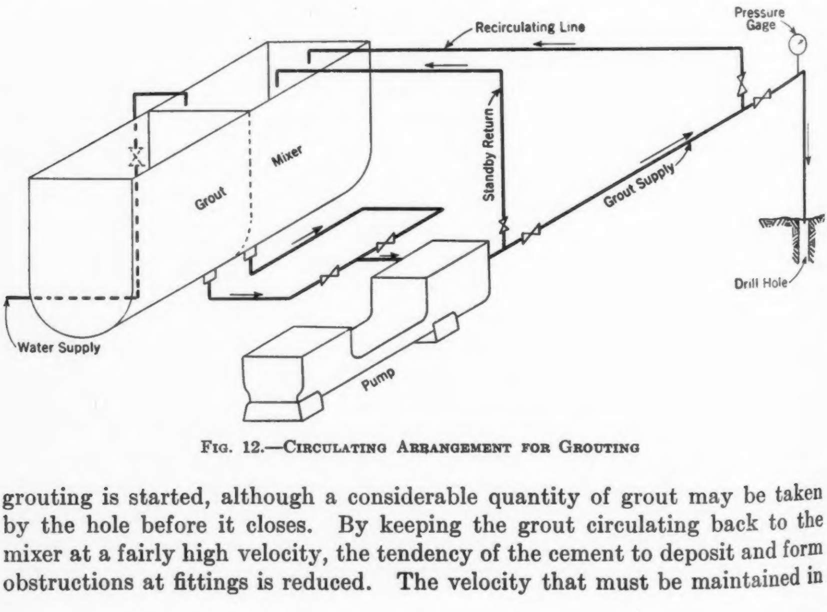


FIG. 12.—CIRCULATING ARRANGEMENT FOR GROUTING

grouting is started, although a considerable quantity of grout may be taken by the hole before it closes. By keeping the grout circulating back to the mixer at a fairly high velocity, the tendency of the cement to deposit and form obstructions at fittings is reduced. The velocity that must be maintained in

order to keep the lines open depends to a great extent upon the prevailing temperature. The modified circulating system also possesses advantages when gradual refusal is occurring as it gives flexibility in controlling the pressure, regardless of the rate at which the hole is taking grout.

The value of a pump for handling grout, as compared with a compressed air chamber, is great. The pump affords additional mixing of the fluid and makes possible a flexibility of control and continuity of flow that is not obtainable with the pneumatic machine. However, the pneumatic machine is superior for handling grout containing sand or sawdust or the other coarse materials which are sometimes injected into large openings.

Tunnels to Seal Large Seams in Abutments

In addition to the grouting of the seams in the abutments, and preceding it, other measures were taken to reduce percolation to a minimum. When the size of the seams indicated the necessity for extra precautions, tunnels were driven parallel to the axis, following the seams back into the abutments. On one level in the east abutment, five tunnels, spaced about 20 ft apart and varying in length from 100 ft to 250 ft, were driven into a seam until it had closed sufficiently so that it was certain that it could be sealed economically with grout. On other levels, not more than one tunnel was driven, with the exception of some very short ones in the west abutment. The longest tunnel was driven under the core wall that extended eastward through the rolled earth fill at the east end of the gravity section. This tunnel was 671 ft long and followed a very large clay-filled seam that started at the dam at El. 965. The seam varied in thickness from a mere contact plane to 40 in., averaging about 18 in., and the clay varied from compact to very loose, containing some openings. It was not unusual to find stretches of 20 to 40 ft where there was no evidence of the seam other than a discolored band in the rock. Such contact areas necessarily exist in any seam to support the weight of the overlying material.

Filling Tunnels.—To fill the tunnels in the abutments, 36-in. core holes were drilled from the surface and concrete was dumped into the holes in 3-cu-yd batches. The long tunnel under the core wall was filled through an 8-in. pipe from the surface, the concrete being forced in with compressed air.

In order to permit the injection of grout into any space resulting from the shrinkage of the concrete in the tunnels, and also to act as an air vent while placing the concrete, two 2-in. pipe headers were run for the full length of each tunnel and 1-in. pipes connected to the headers were extended upward into jackhammer holes, drilled 12 in. vertically into the roof of the tunnel. The 1-in. pipes were left open and were socketed in this manner in an effort to prevent them from filling with mortar from the concrete. High points in the tunnel roof were chosen as the location for these 1-in. branches. In addition to the pipes inserted in holes as described, others were run back into the seam that the tunnel had followed. These were spaced about 10 ft apart and were installed for the purpose of grouting the seam between the tunnels. They were connected to the same header and dry mortar was packed between and

around them where they entered the seam. The arrangement of the grout pipes in one of the tunnels is shown in Fig. 13. After filling the tunnels with concrete, grouting was always deferred as long as possible in order to allow time for shrinkage. After the tunnels had been filled with concrete and grouted,

FIG. 13.—VIEW IN TUNNEL SHOWING GROUT PIPES AND LARGE SEAM

the areas lying between parallel tunnels were drilled from above in the usual manner, and any remaining open spaces were washed and grouted through the holes. Eight tunnels were driven into the east abutment, not including the long one under the core wall, and four were driven into the west abutment.

SEALING CONTRACTION SPACES IN ABUTMENTS

In the vertical or steeply sloping rock faces of the abutments, a water stop of 16-gage copper, 14 in. wide, was embedded 3 ft inside of the upstream face of the dam. Jackhammer holes 10 in. deep were drilled on close centers, and the web between them was broached out to form a groove in the rock into which the copper strip was sealed with a stiff mortar. Half the width of the copper strip was allowed to project out of the groove to be embedded in the concrete.

In addition to the water stops that have been described, grout outlets for filling any space resulting from contraction of the concrete were installed on the vertical faces of the rock. The outlets consisted of matched flanges, so arranged that one flange was anchored in the rock face while the other was held by the concrete. It was intended that shrinkage of the mass concrete should separate the flanges, permitting the entrance of grout to the resulting space between rock and concrete. The flanges were connected to pipes running to the gallery in the dam.

Investigations for Deep Foundation Treatment

When the shallow grouting was completed, the concreting of the structure had progressed sufficiently so that the deep drilling for the curtain grouting from within the gallery could be started. The section through the spillway in Fig. 14 shows the location of these holes, which varied in depth from 60 to

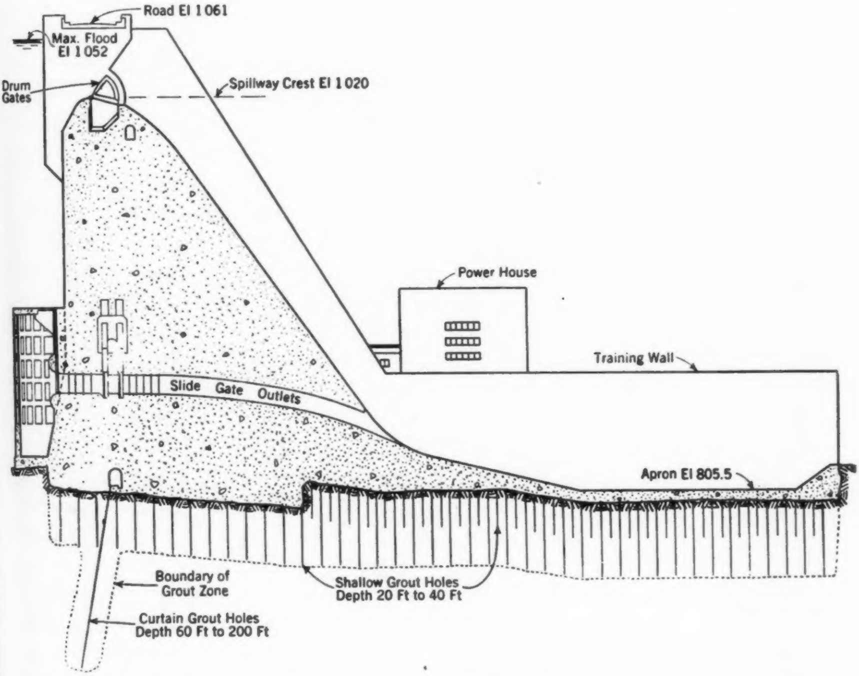


Fig. 14.—Section Through Spillway and Slide Gate Outlets Showing Foundation Grout Holes

200 ft, depending upon the conditions encountered. It may be noted that few of the holes extended to the maximum depth. For this curtain grouting, 5.5-in. shot core holes, on the upstream side of the gallery and sloping upstream at an angle of 10° with the vertical, were drilled on 10-ft centers for the length of the dam. However, before drilling the grout holes, vertical upheaval gage holes, spaced 60 ft apart and driven to a depth 10 ft in excess of that tentatively chosen for the grout holes, were drilled. The purpose of drilling these holes in advance was primarily to obtain accurate and detailed information concerning the location of seams. By combining the logs of the drilling and the exploration records of the holes, it was possible, before drilling for grouting was started, to plot a very useful section through the rock underlying the dam, showing the location, thickness, and general characteristics of all the major seams. When developed for the full length of the dam, this section was used as a basis for planning the original program of curtain grouting. As additional

information was secured by drilling the grout holes, the program was modified to meet new conditions.

Grouping of Holes.—The work of drilling and grouting the holes 10 ft apart in the galleries was divided into three parts. First, groups of three holes on 100-ft centers between groups were drilled, washed, and grouted. Then groups of three holes half-way between the first groups were treated similarly. This left space for two holes between the first and second groups and these were drilled last. By following this plan, an effect somewhat comparable to stage grouting was obtained, although without the delays incidental to that method. The concentrated washing of the seams that was allowed by grouping the holes to secure interconnection between them was of considerable advantage. The washing of the first groups tended, of course, to remove some unsound material from the surrounding area, and the grouting of these holes effected partial consolidation of the area outside of the group. The treatment through the second groups extended to, and possibly overlapped, the consolidation effected from the first. The final washing and grouting through the two-hole groups in the partly consolidated areas was designed to complete the formation of the grouted curtain.

Exploring Deep Holes.—For exploring the 5.5-in. inclined holes in the gallery, the device shown in Fig. 15 was developed. It was essentially a refinement of the "Feeler" (Fig. 4) used in shallow holes, and consisted of a wire centering cage containing a pair of spring-actuated legs so adjusted that an electrical circuit was closed, flashing a light for the observer, when either leg moved outside of the travel limits fixed by a hole of normal size. The cage was suspended on a graduated $\frac{1}{8}$-in. bronze cable with an insulated single conductor taped parallel and close to it. The bronze cable served as a conductor also. A latch held the legs in the closed position until the bottom of the hole was reached, when they were released automatically and were free to open into any seam encountered as the device was pulled upward. Each hole was explored twice to reduce the risk of missing seams. The device was very effective for locating seams, and the ease of handling afforded by its lesser weight, plus the fact that it was probably superior at the greater depths, gave it a marked advantage over the original explorer.

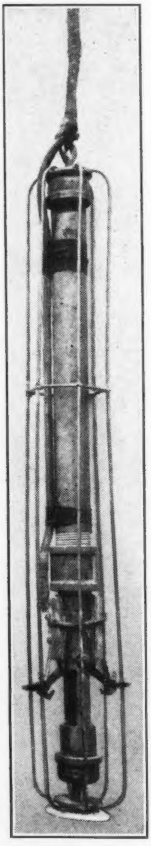


FIG. 15.—ELECTRICAL EXPLORER

WASHING THROUGH DEEP HOLES

Following the location of the seams in any group of holes with the exploring device, each seam was washed individually by means of the pneumatic expander shown in Fig. 16. This expander consisted of two sleeves of heavy gum rubber, separated by a piece of perforated pipe, and was suspended from the surface

on a marked $\frac{1}{4}$-in. steel cable, with a parallel hose carrying the washing fluid. A separate $\frac{1}{4}$-in. hose supplied air to expand the rubber sleeves, and both hoses were lashed to the cable at about 10-ft intervals. By centering the perforated pipe on a seam and expanding the sleeves, the entire supply of washing fluid could be directed into the one seam. This pneumatic type of expander

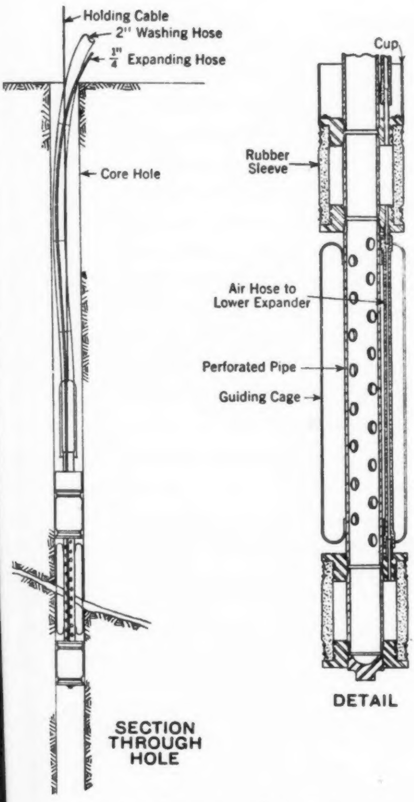


Fig. 16.—Pneumatic Expander

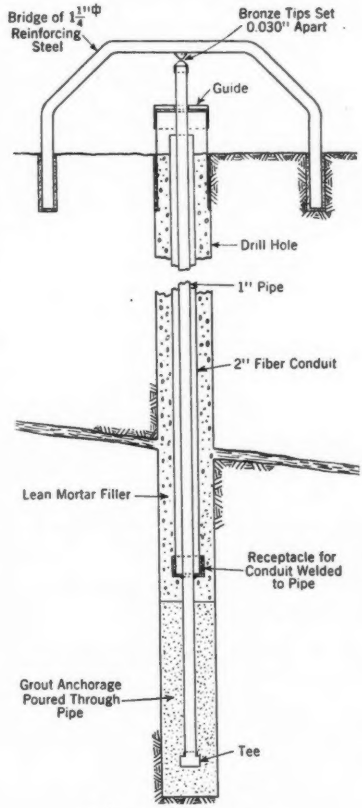


Fig. 17.—Upheaval Gage

possesses decided advantages where the head-room is limited, as in the grouting galleries, or when the holes are very deep. Where the head-room is limited, the pipe that supports the mechanical type must be cut in short sections, making it awkward to handle; and in deep holes, the increased weight adds to the difficulties. A minor detail is the cup that is mounted on top to catch fragments of rock that fall from seams and shattered zones. If not caught, these fragments will wedge in the space between the expander and the wall of the hole and occasionally result in the loss of the equipment. Sufficient air pressure is required to expand the rubber against the total head of the washing water, at the depth of the seam being washed, if leakage is to be prevented.

A crew well trained in handling, and familiar with the limitations of the device in loose, crumbling, and otherwise unsound rock, is necessary to minimize trouble.

In washing a group of holes, the expander was always placed first on the lowest seam. The initial quantity of water that the seam would take at the washing pressure was then measured by passing it through a calibrated orifice assembly which contained orifices of different sizes to cover a wide range of flow. While washing, checks were made at regular intervals to learn whether the seam was opening as the result of the removal of material. Air and water were used, mixed and separately, to loosen the material, and the expansion of the air brought large quantities of muck to the surface from the connected holes. It was necessary, at intervals, to blow the nearby holes clear of muck that was washed into them and settled below the seam from which it was being washed. The length of time to be spent on each seam was judged by the change in the color of the water overflowing from connected holes. If there was no overflow from other holes, the rate of consumption of the wash water was measured until no increase was noted, when the expander was moved to the next seam above. As the overflow from any connected hole cleared up, this hole was plugged at the surface in order to force a passage to the other holes in the group. In this manner, each seam in each hole was thoroughly washed. Frequently, seams were found to be so tight that they would take no measurable quantity of water and the expander was moved when this fact was established by a reasonable trial. However, it was not uncommon to find that a seam, initially offering great resistance to flow, would open up and take large quantities of water after pressure had been maintained on it for some time. After the washing was completed, and just before the grouting was started, the holes were blown out with air to remove all muck.

Great stress was always placed upon the importance of thorough washing of the seams, as it was felt that the effectiveness of the grouting depended almost wholly upon the success achieved in removing the unsound material. The pressure of the wash water at the surface did not, in general, exceed 100 lb per sq in.

Water-Cement Ratio.—The water-cement ratio of the grout was determined in advance for each group, or for any individual holes not connected with others, by measuring the quantity of water which the holes would take at the washing pressure. For very tight holes the water-cement ratio was 3.0; for holes offering a fair degree of resistance, 1.5; and for holes that were very open, 0.66. What was termed the normal mix had a water-cement ratio of 1.0. This normal mix was used on holes that offered an average degree of resistance, and it possessed sufficient fluidity to be satisfactorily handled by the pumps. When the ratio was reduced below 0.66, the consistency of the grout was so great that trouble was experienced in handling it through long pipe lines. Extremely accurate control of the water was not considered necessary in view of the fact that much of the grout was pumped into water-filled seams.

Curtain Grouting

After a group of holes had been washed and blown free of muck as described, short single expanders were placed in each hole. If connection with another group had developed during the washing, expanders were also placed in the holes of the other group. When a free connection between groups existed, every effort was made to grout them simultaneously with separate pumps. However, this was not always practicable and, in such cases, the lines were extended from one pump to both groups. Pumping would be started on one hole of a group at a rate of between 100 and 200 cu ft of cement per hr. When grout flowed from the other holes of the group, possibly many hours later, the overflow was stopped by means of valves on the expanders, and pumping was continued until overflow from the connected group resulted. The latter group was then closed off too, and the pressure on both groups was measured with the pump running. If the connected group showed appreciably less pressure than the group into which grout was being pumped, the flow of grout was diverted to the former, and from that time to refusal, the flow was changed back and forth between the two groups to maintain approximately the same pressure on both. When overflow failed to develop from a group with which interconnection was known to exist, the holes in this group were sampled at intervals by sounding with an open bottle to learn whether grout was leaking in. If the soundings indicated that it was coming in at a very low rate, the hole was kept open by blowing it out until a pump could be connected to it.

Pumping Rate.—On holes that offered no initial resistance to the inflow of grout, it was necessary to exercise judgment in determining the rate of pumping. In grouting seams of wide extent, pumping at a high rate may result in forcing large quantities of grout into remote regions where it is wasted in so far as the objective is concerned. A better effect might be obtained more economically by the judicious use of a much smaller quantity of thick grout, pumped slowly.

Upheaval Gages.—As a result of the great area covered by the seams underlying the foundation, the danger of raising the structure by the use of excessive pressures was a very real one, and upheaval gages were installed in the grouting gallery at 60-ft intervals for the length of the dam. The gage (see Fig. 17) consisted of a piece of 1-in. pipe with its lower end anchored by grout in the bottom of a vertical hole that had been drilled 10 ft deeper than the nearby grout holes. Above the anchorage, the pipe was encased in 2-in., asphalt-dipped, fiber conduit. After the pipe had been anchored by pouring grout through it, it was held in a vertical position by maintaining a strain on the top while the hole was filled with lean, coarse mortar. The strain was held until the mortar had set, thus assuring that the pipe would be restrained against excessive deflection as the result of carrying its own weight as a column. Across the top of the pipe, and set in the gallery floor, a bridge of 1.25-in. square reinforcing steel carried a bronze tip that was set approximately 0.030 in. from a similar bronze tip on the pipe. The gap between the tips was measured with a thickness gage at frequent intervals during grouting, and, if a progressive

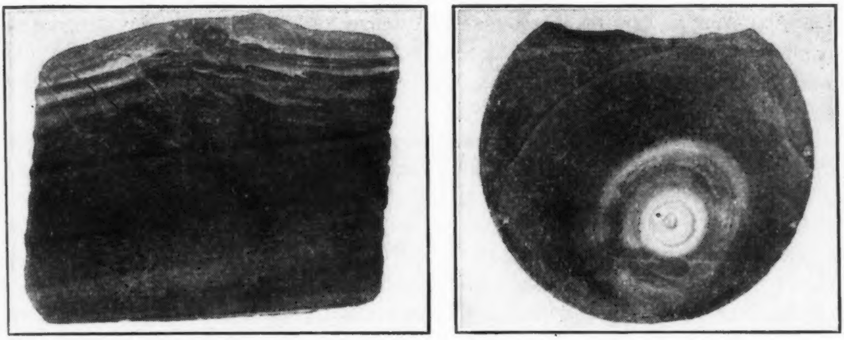
increase of as much as 0.010 in. occurred, it was construed as a warning that the safe pressure was being exceeded. When this occurred, the header was removed from the hole and the grout was allowed to flow out, the uplift gage being observed constantly to ascertain when settlement of the structure had ceased. When the original position of the structure had been assumed, or settlement had ceased, grouting was resumed at a reduced pressure. This gage was designed by the Bureau of Reclamation at Denver, Colo.

Spacing and Depth of Holes.—In general, the holes for the curtain grouting were drilled 10 ft apart, but it was found desirable to reduce this spacing in some particularly bad areas to obtain a tight seal. Although this spacing might seem unnecessarily close to one who visualized extensive open seams through which grout could flow without interference, this condition did not exist in fact. In the writer's opinion, a plan view of any seam would have revealed that between 30% and 60% of the total area consisted of contact areas where no opening existed, and which, when penetrated by a drill hole, would not permit grout to flow into the seam. Thus, many of the holes drilled were not effective for grouting in some of the seams penetrated. This conclusion is based on observations made in a number of tunnels driven into seams, where it was noticeable that the openings frequently vanished entirely, only to reappear further along.

The locations of the principal seams were learned before the grout curtain holes were drilled, and therefore it was possible to determine, in advance, the depth to which the curtain should extend. To care for any local irregularities, holes were drilled in general 10 ft below the lowest seam known to underlie the location, or deeper if it was found that treatment was required at lower elevations.

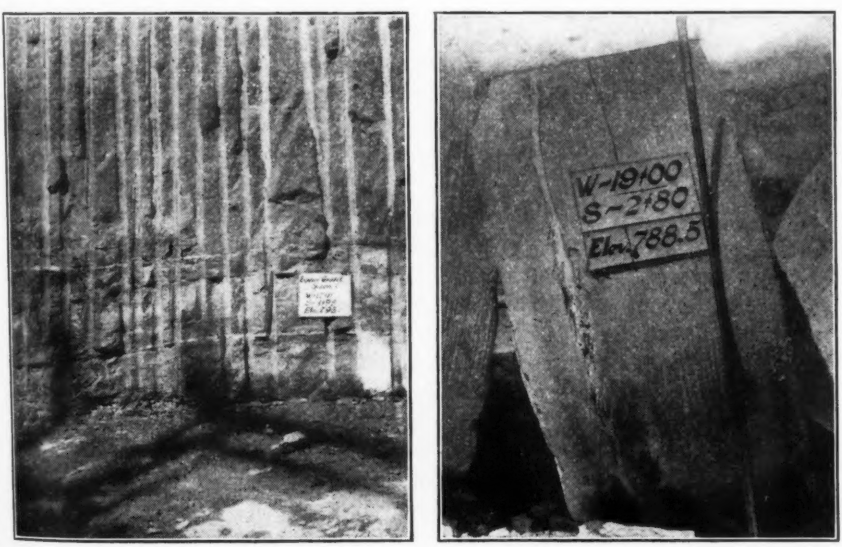
Action of Grout.—In any grouting operation, great uncertainty prevails at all times as to just what is happening beneath the surface and as to whether the desired results are being obtained. The question of achieving economies by variations in methods arises constantly and must be judged with regard to the adverse effect upon the quality of the work that may result. If a uniformly open seam is pictured, the process is simple to analyze, but when one considers the infinite number of variations that may exist to defeat a preconceived notion, the uncertainty increases. Seams contain, in themselves, numerous channels, the easiest of which will be followed by the fluid grout, possibly for days. This fluid laps and splashes to form a hard coating over sand, clay, and unsound materials that may border the channel and, in addition, slowly deposits and builds over narrow passages leading from the channel. By the time that sufficient solid material has been deposited in the channel to offer enough resistance to flow to cause the grout to seek other passageways, these openings have been sealed over and will not permit the entrance of grout. The result of pumping a large quantity of grout into one hole may thus have been to fill one channel for a great distance without making much improvement on the condition of the general area. In some such manner, it frequently has happened that open passages have been left in a line of holes that were not spaced closely enough.

From observations made on numerous cores drilled from grouted seams and grouted holes, it appears that, in seams, the solid grout builds up slowly in layers as the cement is deposited. It is probable that this deposition begins when the velocity of the grout is reduced as it spreads out after leaving the



(*a*) Typical Seam (*b*) Typical Core

FIG. 18.—POLISHED SECTIONS SHOWING CHARACTERISTIC DEPOSITION OF GROUT



(*a*) Exposed by Excavation (*b*) In Large Core

FIG. 19.—TYPICAL GROUT SEAMS

drill hole. As continued deposition results in constriction of the passage, the cement is deposited farther and farther from the hole. Chemical affinity between the particles of cement is also an important factor in this building up of the solid material. Evidence of this fact may be seen in cores removed from redrilled grouted holes, where it frequently may be observed that the cement

has built up in concentric layers, gradually closing in and reducing the size of the passage. The possibility should not be overlooked that the drill hole may close before the seam plugs. The final closure probably results from the tendency of the cement particles in suspension to cohere to the deposited cement, gradually closing off the passageway. Many things may happen to make the cause of this final closure obscure and uncertain. Samples cored from both seams and grouted holes, showing the manner in which the grout was deposited, are shown in Fig. 18. Fig. 19 shows some grout-filled seams that were exposed after grouting.

Pressures.—In general, a refusal pressure of 150 lb per sq in. was required for the curtain grouting, although it was occasionally necessary to reduce the pressure in order to avoid lifting the structure. As this work was done from the grouting gallery in the dam, after the concrete had been poured to a considerable height, it was necessary to exercise great care to prevent damaging the structure by distorting the individual blocks.

Leakage.—Careful observations made regularly since the reservoir has been filled have not disclosed any evidence of leakage. With a full reservoir, the total flow measured in the stream bed at a point approximately 1 mile below the dam was 2 cu ft per sec. This quantity included the flow from a number of springs that existed before the dam was built. Test holes drilled to penetrate the grouted curtain at intervals of approximately 100 ft throughout the length of the dam have revealed a degree of tightness that was unexpected, the total flow from all holes being approximately 0.03 cu ft per sec. Nineteen drain holes, drilled in the spillway apron to a depth of 15 ft below the grouted zone, discharge a total of 0.55 cu ft per sec.

Sealing of Rock Under Core Wall

From the east end of the gravity section of the dam, a reinforced concrete core wall was extended 584 ft through the rolled earth fill and overburden, and the grouting program along this core wall differed in no important respect from the methods used under the dam. It was necessary that the core drilling be done through overburden that varied from 50 to 100 ft in thickness. A large clay-filled seam (Fig. 20) was found to underlie this area at El. 965, and it was decided that a concrete-filled tunnel that followed the seam, directly beneath the core wall, would form the most economical and effective seal. Since it would have resulted in waste and unneeded duplication to have grouted this seam in addition to plugging it with concrete, the grouting was divided into two zones, one above the seam and one below it. Following this plan, the first drilling was stopped when the holes penetrated the seam at El. 965. Before grouting, thick dry mortar was poured into the holes to form a plug at the bottom that would prevent the passage of grout to the seam. The holes could not be stopped just short of penetration as the exact elevation at which the seam would be found was not known in advance. The seams in the upper zone were washed and grouted to refusal at a pressure of 75 lb per sq in.

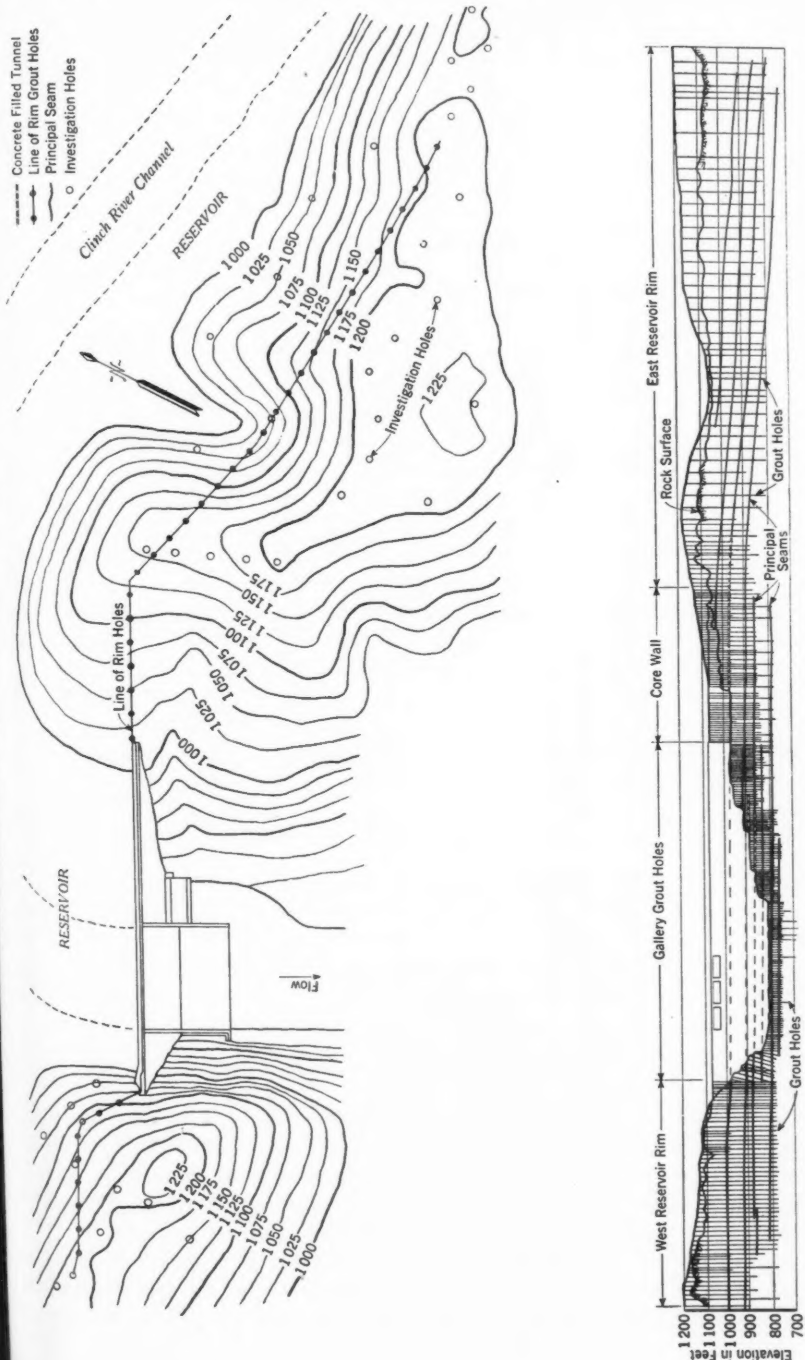


FIG. 20.—TOPOGRAPHY AND SECTION OF GROUTED AREAS

Jaw Packers.—As the normal reservoir level was at El. 1,020, it was unnecessary to grout rock above this level, and packers were used to control the limits of the grouting. The elevation of the overburden from which the drilling was done varied from 1,060 to 1,170, and the elevation of the surface of the rock under this overburden varied from 965 to 1,085. The expanding, tapered-body, rubber-sleeve type of packer shown in Fig. 21 was developed for

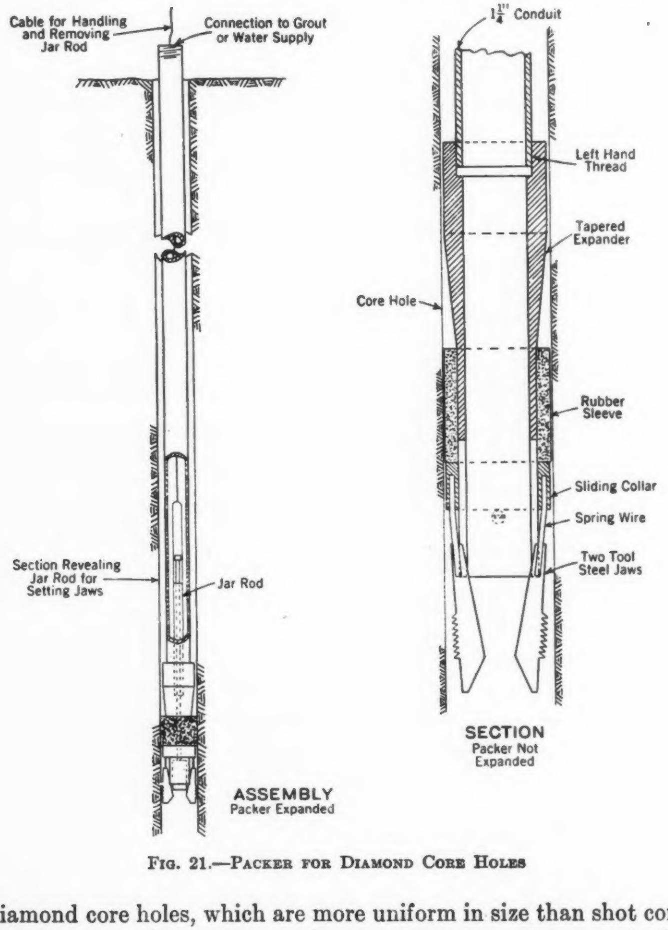


FIG. 21.—PACKER FOR DIAMOND CORE HOLES

use in diamond core holes, which are more uniform in size than shot core holes. In general, however, this packer was found to be quite satisfactory in the latter type of hole also. Cup leather packers were used when a hole was too large to permit the rubber sleeve of the jaw type to jam against the wall. The packer was always set at the elevation below which it was desired to grout.

In practice, the jaw packer was set by lowering it to position on $1\frac{1}{4}$-in. electrical conduit. Conduit was chosen in preference to pipe by reason of the ease of handling afforded by the short joints of uniform length and by the

lighter weight. Having placed the packer at the desired elevation, a weight called a jar rod was lowered on a line until it rested upon the tapered jaws of the packer. The jar rod was made with a sliding body so that it could be driven quite forcefully against the jaws by raising and releasing the body with the handling line. By driving in this manner, the teeth on the packer jaws were forced to grip the wall of the hole and were held in that position by the weight of the jar rod. The conduit was then released slowly and its downward motion forced the rubber sleeve to expand upon the tapered body of the packer until it jammed against the wall of the hole and formed a seal. The jar rod was then withdrawn, as the friction of the rubber against the side of the hole was usually sufficient to support the column of pipe. When the friction was not sufficient, the weight was supported by a clamp that rested on the top of the casing. When grouting was completed, the packer was removed simply by pulling it out of the hole and was ready for use again immediately. To preclude the possibility of losing the conduit, should the packer become hung in any manner, the connection where it joined the packer was made with a left-hand thread so that the full length of conduit could be released by turning it clockwise.

When the upper zone had been grouted, a minimum of 24 hr was allowed to elapse before drilling was resumed to carry the hole to the final depth. Following the exploration and washing of the lower part of the hole, a packer was set immediately below the seam and the final grouting was completed at a pressure of 150 lb per sq in.

The labor cost of a grouting crew was as follows: 1 foreman at $1.00 per hr; 1 pump operator at $0.75 per hr; and 5 laborers at $0.45 per hr.

For grouting the dam foundations, 202,770 cu ft of cement grout at $1.521 per cu ft was required. The latter figure includes the cost of washing at $0.21 per cu ft of grout injected and the cost of pipe and fittings at $0.05 per cu ft of grout injected.

Shot Core Drilling

For the curtain grouting of the dam and core wall, both 3-in. and 5.5-in. shot core holes were used. Genuine economy was obtained by using the smaller size only when it was necessary to drive casing through overburden. Although the initial investment in the smaller drilling tools was less and the shot consumption was less, the wear and consequent replacement was much greater and tended to counteract any economies that might have seemed apparent. Breakage of tools in the hole, with the resulting loss of time, was much more common with the smaller tools. The 5.5-in. holes served more effectively for exploration and seam washing and, in concrete or rock, cost little more than the 3-in. holes. The difference in cost of $1.026 per ft indicated in Table 1 is accounted for by the favorable effect of the large proportion of overburden drilling in the smaller size. Contributing to the foregoing unit cost was the labor cost of the typical core-drill crew, as follows: 1 driller at $1.00 per hr; 1 helper at $0.60 per hr; and 1 foreman at $1.25 per hr. The foreman supervised all drill crews.

Improvements in the rate of drilling, with a resulting reduction in costs, were secured by posting a daily record of each driller's footage and sum-

marizing this at the end of the month to show the high men for the period. This was very effective in promoting a spirit of competition among drill operators and among shift foremen. Average drilling rates, costs, and other data are contained in Table 1.

TABLE 1.—DRILLING DATA

Drill Hole		Extent of Drilling, in Linear Feet of Penetration					Unit Costs, in Dollars Per Linear Foot	
Type	Diameter, in inches	Overburden	Rock	Total	Average depth	Average drilling rate, in feet per hour	Overburden	Rock
Wagon	2½	0	149,308	149,308	32	13.00	....	0.408
Shot core	3	4,532	15,363	19,895	245	1.28	....	2.634[b]
Shot core	5½	0	32,248	32,248	100	1.20	....	3.66
Diamond core[a]	2 1/16	1,935	7,721	9,656	360	1.65	2.25	2.25
Diamond core[a]	2 5/16	4,054	26,344	30,398	320	1.50	2.50	2.75
Well[a]	6¼	844	3,713	4,557	325	1.00	2.40	2.40

[a] Contracted. [b] Composite cost of overburden and rock.

RESERVOIR RIM INVESTIGATION

The geological survey made preliminary to the selection of the site revealed that the rim of the reservoir for distances of 9 miles from the east abutment and 5 miles from the west abutment would probably allow some leakage of water from the reservoir because of the geological characteristics of the formation. The cost of treating, or even of thoroughly investigating such a lengthy stretch of rim, would obviously have been prohibitive, and it was decided to confine the treatment to the narrow and definitely permeable parts adjacent to the abutments.

At the beginning of the rim investigation and treatment, the elevation of the water table underlying the areas in question was used as the criterion by which the permeability of the ridge was judged. That is, a high water table was interpreted as indicating that the rock beneath must necessarily be tight in order to support the water, and that, conversely, the rock above a low water table was sufficiently permeable to allow drainage of the ground water. Investigations must be sufficiently thorough to ascertain that a perched water table is not mistaken for the real water table. The first step, after determining the general areas to be investigated, was to drill $2\frac{1}{16}$-in. diamond core holes approximately 200 ft apart on a line along the top of the ridge and to a depth reaching well below the bed of the river. The general layout of the investigation holes may be seen in Fig. 20. These holes were sounded daily for a sufficient length of time to determine the relation of the elevation of the water table to variations in the level of the water in the river. Where apparently open or pervious areas were found, as judged by the sensitivity of the table to changes in the elevation of the river water, other holes were drilled on each side of the ridge to establish a gradient and to ascertain that no tight barrier existed in that part of the ridge away from the reservoir. When the areas had

been drilled as shown, daily soundings were made, extending over a period during which sufficient variation in river level occurred so that the characteristics of the holes were established definitely. By studying the hydrographs of the holes, it could be seen that from the beginning some followed closely every change in the elevation of the river water. Other holes never did fluctuate, whereas still others did not become active until the water rose to higher elevations, indicating the certainty that an open seam existed at the elevation at which the activity started. From the information obtained by sounding the holes, a contour map of the water table underlying the areas adjoining the abutments on each side of the river was made. This map, in conjunction with contour maps of the ledge rock and of the ground surface, was used to determine the location of the line of grout holes.

Diamond Core Holes.—During the drilling of the diamond core exploration holes, a detailed log of all findings, such as seams, core losses, loss of water, peculiarities of the rock, and any other pertinent information, was kept. However, the reliability of a large part of such information depends to a great extent upon the interest, vigilance, experience, and general ability of the drill operators, as well as upon the type and mechanical condition of the equipment. When using a screw feed machine, it is necessary (when the action of the machine indicates that a seam has been encountered) to release the chuck in order to measure the thickness of the seam accurately. This necessarily causes a delay in the contractor's operations, and too great a desire for progress will result in many seams being overlooked. Inattention and lack of interest on the part of the driller gives the same unsatisfactory results. The use of a double-tube core barrel, as compared with the single-tube barrel, reduces the core losses appreciably. A study of the records of the operations of two contractors, one using the former and one the latter for drilling in the same formation, revealed that the core loss was 5.9% with the double-tube barrel as compared with 24.9% with the single-tube. When drilling for grouting, the use of heavy compounds for lubricating the drill rods should be prohibited, as it is possible that this "rod dope" mixes with the cuttings to form a compound that seals small seams effectively against the entry of grout. Experience indicates that a diamond core drill, equipped with hydraulic feed, and using a double-tube core barrel, gives the most reliable results obtainable with small borings when information concerning subsurface openings is desired.

Rim Grouting

Having determined the approximate location of the water table and of the rock surface in the areas in question and having obtained a general idea as to the frequency, location, and size of seams, it was next necessary to locate the lines upon which the grout holes were to be drilled. On the east side, in consideration of the length of the permeable portion, the distance of the region of low water table from the dam proper, and the character of seams and thickness of ridge, El. 1,020 (spillway crest) was set as the upper limit of the grouting, with holes spaced 50 ft apart. On the west side, because of the more cavernous nature of the rock, the close proximity to the abutment of the region of low water table, and the certain existence of large extensive seams, the upper

limit of the grouting was raised to El. 1,050 through the most permeable portion, and the spacing of the holes was reduced to 20 ft. On both sides of the river, the holes were drilled to a depth that reached below river bed.

Location of Holes.—In locating the lines of grout holes then, it was necessary that the surface of the rock lie above the elevation of the upper limit of the grouting and desirable, in the interest of economy, to have the elevation of the ground surface as low as possible in order to minimize the amount of drilling. However, it was necessary to guard against apparent savings in depth secured by circumventing hills, as the resulting increase in the number of holes tended to offset any gains.

Since practically all of the holes were drilled from an elevation well above the upper limit of the grouting, it was desirable to avoid waste of the grout by confining it to the regions where it was needed. For this purpose, the previously described jaw packer shown in Fig. 21 was used.

On the west side, the line of holes was turned sharply upstream from the extended axis of the dam to avoid a large cave at El. 980. This cave opened on the bluff a short distance below the axis and extended back into the hill and across the axis. By grouting upstream from the cave, the cutoff was obtained more economically than it could have been secured by removing the loose material and filling the cave with concrete.

Water Tests.—After the packer was set at the desired elevation in a hole, a water test was made in an effort to gain some idea as to the quantity of grout that would be taken. The usual range of test pressures varied between 25 and 50 lb per sq in. This test was of value in that it was generally true that a tight hole would take little grout, whereas a very open hole could be expected to take large quantities. No rational basis for estimating the actual quantity of grout that would be required was determined.

Seams.—The elevation of the water table was considered highly important in determining the locations of the permeable portions of the rim, but, as more information concerning the location and size of the various seams was obtained from the drilling of the grout holes, the problem became more and more a matter of following and checking these seams carefully, regardless of the position of the water table. The position of the water table was obviously of little importance if seams (which, under greater head, would serve as drains) existed below it. Having determined the dip and strike of the rock accurately, it was possible to compute the approximate elevation at which a given seam should be found in any hole. By exercising unusual care when approaching the location at which the seam was expected to occur, the driller frequently could tell when it was penetrated. When no opening was found, the core loss often indicated the existence of unsound rock. Loss of drill water was also construed as a definite indication of an open seam.

Fig. 20 shows the seams that were traced from the ends of the dam through the rim on each side. The greater apparent angle of dip on the east side of the river resulted from a bend in the line of holes that placed the line nearer normal to the strike. The grouted curtain was ended on each side where the water table was found well above reservoir level. Although it was realized that it

was quite possible that leakage would later occur through the rim, both within and beyond the grouted portions, it was felt that the expense of investigating and treating the entire part of the length that was questionable was not justified. It was decided, therefore, that the necessity for any additional treatment would be determined after the reservoir had been filled for some time and observations had revealed the seriousness of any leaks that might develop. Should leaks appear, drilling could be done with a full reservoir so that dye could be placed in the holes to determine when the path of the flow had been penetrated. In a number of cases, during the period of foundation treatment, fluorescein was used successfully for tracing subsurface flows. This dye may be traced in a very diluted solution if white porcelain containers are used for sampling the water.

The grouting of the reservoir rim differed from that of the dam foundations in several major respects. No attempt was made to wash unsound material from the seams, as it was felt that the small size of the $2\frac{5}{16}$-in. holes, spaced so widely, precluded the possibility of applying a sufficient volume of water to be effective. The objective was not a complete replacement of unsound material with grout, but a consolidation of this material by penetrating and filling the interstices with hard grout put in under sufficient pressure to compact the loose material in the seams. It was known that the openings in the rim were of such size and extent that, by maintaining a high rate of pumping, the grout could be forced to travel unneeded distances, increasing the consumption of material appreciably. Accordingly, the rate of pumping was usually limited so that not more than 80 to 100 cu ft per hr of solid material was handled by one pump.

Pressures.—In general, the refusal pressure for this work was limited to 25 lb per sq in. at the surface. As applied to holes of different depths, this was not an altogether consistent practice, as some variation in the pressure at the bottoms of the holes resulted. However, it was not thought necessary to use high pressures to obtain effective consolidation, although it was desirable to have some pressure on those seams which were near the top of the holes. The refusal pressure was set arbitrarily at 25 lb per sq in. at the surface in the belief that this pressure would afford the desired consolidation in the upper seams without causing undue waste in the lower regions. Since a general tightening of the rim was evidenced as the program neared completion, the pressure was sometimes raised in order to consolidate areas that were known to be unusually bad.

All studies following the preliminary investigations had tended to support the belief that large quantities of material would be required to consolidate the rim to the degree desired. These studies consisted of water tests of exploration holes and daily observation of the water levels in these holes to determine their sensitivity to fluctuations of the river level and to rainfall. Holes in which the elevation of the water always bore a close relation to the elevation of the river water, and that were little affected by rainfall, were considered as lying in open or permeable territory. Conversely, holes that, apparently, were not affected by river changes, and which were noticeably affected by rainfall, were considered as lying in tight rock.

Use of Rock Flour

With the thought that large quantities of material would be required and that a fine inert material might be used economically with cement to form satisfactory grout for rim tightening, a settling basin was built to collect rock flour from the sand classifier of the aggregate plant.

Preliminary Tests.—To determine the suitability of the rock flour cement mixture for grouting, preliminary field tests were made in an attempt to learn the characteristics of the resulting product. Samples that were taken from the mixer were observed to set and harden at a rate that indicated considerable retardation when compared with regular cement grout. Pumping tests proved that the characteristics of the mixture differed from those of cement grout to a degree that would make it desirable to revise the grouting program if its use was adopted, and established the necessity for laboratory tests supplemented by additional investigations in the field.

Laboratory Tests.—The laboratory tests indicated that the material was finer than cement, and a moisture-weight determination was made for use in proportioning mixes. The time-of-set data were perhaps the most interesting and pertinent as regards the determination of the suitability of the material for grouting, and it was definitely proved that the addition of rock flour had a retarding effect upon the setting time of the mixture.

Importance of Setting Time.—It may safely be assumed that, in grouting extensively seamed rock at a fixed rate of pumping, the area that will be covered is dependent to a large extent upon the setting time of the cementing material. By the use of a slow-setting material, the distance traveled by the fluid may be increased so that areas completely outside of the region that it is desired to treat will be grouted and the quantity of material necessary to effect consolidation

TABLE 2.—Normal Cement and Rock Flour

Test No.	Mix by Volume			Time of Setting, in Hours and Minutes			Test No.	Mix by Volume			Time of Setting, in Hours and Minutes		
	Cement	Rock flour	Water	Initial set	Final set	Soundness[c]		Cement	Rock flour	Water	Initial set	Final set	Soundness[c]
1	1	...	...[a]	2:10	5:00	OK	9	2	1	...[a]	2:50	6:35	OK
2	1	...	...[ab]	0:40	2:00	OK	10	2	1	...[ab]	1:45	3:25	OK
3	1	...	1.5	36:00	...	...	11	2	1	1.5	11:00	22:00	OK
4	1	...	1.5[b]	24:00	...	...	12	2	1	1.5[b]	4:05	10:30	OK
5	1	1	...[a]	3:15	6:50	OK	13	1	2	...[a]	4:30	6:45	OK
6	1	1	...[ab]	2:45	5:00	OK	14	1	2	...[ab]	2:15	4:20	OK
7	1	1	1.5	30:00	...	...	15	1	2	1.5	16:00	36:00	OK
8	1	1	1.5[b]	28:00	...	...	16	1	2	1.5[b]	10:00	18:00	OK

[a] Normal consistency (water 23% by weight of cement, or cement and rock flour). [b] $CaCl_2$, 3% by weight of cement, was added to the water. [c] Tests Nos. 3, 4, 7, and 8 did not attain final set in 6½ days. These mixes have an excess of water covering sample.

increased appreciably. Obviously, even if the unit cost of the grout was reduced by the use of a cheap, inert material to replace part of the cement, the total cost of treating a given area might be increased if the quantity of material consumed was appreciably larger.

Addition of Calcium Chloride.—For these reasons, tests were made to determine the effect upon setting time secured by the addition of calcium chloride to the grout. The results of the complete series of tests are given in Table 2. It was found that the addition of this chemical had a tendency to correct the undesirable qualities of the rock flour mixture. It was learned that, by the addition of 3% of calcium chloride by weight of the cement, the set could be accelerated to a degree that would counteract, to a marked extent, the retardation resulting from the use of rock flour. The resulting product possessed characteristics of pumping and handling similar to those of the regular portland cement grout. In appearance, the product was little different from the regular grout, being dense and apparently impermeable, although not quite as hard. Specimens cored from grout-filled seams, after the rim grouting was under way, failed in compression at 2,000 lb per sq in. at an approximate age of 45 days.

Trial Run.—As a further step toward learning more of the properties of the rock flour grout, a test was conducted in which an effort was made to simulate the conditions surrounding the actual grouting of a seam. Four circular concrete slabs, ground to a flat uniform surface on one side, were matched in pairs, so that simultaneous tests could be run with the two types of grout. Temperature differences that might otherwise have affected the comparison were thus eliminated. Metal shims held the two slabs apart $\frac{1}{16}$ in. to form an artificial seam, and the grout entered this space through a 1-in. pipe passing through the center of the upper slab. A barrel in which the level of the grout was held constant by an overflow pipe, and into which the grout pump discharged, served to maintain a constant static head on the slabs. The pressure at the point where the grout entered the $\frac{1}{16}$-in. space was measured constantly during the test by means of a manometer. Throughout the duration of the test, and without interrupting the continuity of flow, the old grout was replaced by fresh grout at frequent intervals. A record of the temperatures of the fluids revealed that consistently higher temperatures prevailed in the regular cement grout. A number of tests were made but the full collection of data is not included in this paper. However, Table 3 summarizes the results obtained,

TABLE 3.—SUMMARY OF RESULTS OF ROCK FLOUR CEMENT GROUT TESTS

Grout mixture[a]	Hours required to grout space between slabs	Ratio of grouting time
2 cement: 2.5 water	19.25	1.00
1 cement: 1 rock flour: 2.5 water	64.30	3.34
1 cement: 1 rock flour: 2.5 water: 3% $CaCl_2$	30.30	1.57
2 cement: 2.5 water: 3% $CaCl_2$	9.50	0.49

[a] Proportion of water by volume; and proportion of calcium chloride ($CaCl_2$) refers to weight of cement.

and it is interesting to note that the addition of 3% of calcium chloride appreciably accelerated the setting of the rock flour cement mixture. Specimens to test for compressive strength were obtained from the mixer in the field, the mix being 1 part cement and 1 part rock flour. The water-cement ratio was 1.0. After 14 days the cylinders failed at an average of about 600 lb per sq in.

The information obtained from these preliminary tests made it appear that considerable economy could be realized by using the rock flour cement grout

containing calcium chloride, and a mixture containing equal proportions of cement and rock flour was adopted for use.

Handling Rock Flour.—It was learned after a trial run that the rock flour contained a quantity of fine clay that failed to disintegrate in the course of mixing in the regular manner. It also contained a small proportion of coarse material that made screening desirable. After some experimentation, it was found that the most satisfactory method of disintegrating the material was by means of a separate, mechanically-agitated mixer from which the rock flour passed in solution, through a screen, to the grout mixer. The mixers were very similar to those used for grout. The rock flour was hauled from the collecting basin as it was needed and did not have an opportunity to dry. The colloidal nature of the material apparently prevented drainage and, even in dry weather, it was necessary only to vibrate the apparently firm surface to reduce it to a jelly-like mass. For proportioning, the rock flour was measured by counting the number of shovels as the material was placed in the mixer, with an occasional check with a measuring box. Although this method seems crude, the nature of the material, its cohesiveness and resistance to breaking down into a granular form, made any other practical method very slow and difficult.

For the reservoir rim, 257,736 cu ft of rock flour cement grout at $0.535 was required.

Sand for Grouting.—During the preliminary grouting for sealing the first cofferdam, an attempt was made to obtain economy by mixing sand with the grout. The sand was placed in the mixer in an effort to handle it in the regular manner and, after a brief time, the pump and lines were plugged solidly. After this unsatisfactory trial, the use of this material was not attempted again until it became necessary to grout some holes in the rim that were known to penetrate unusually large openings. In this case, the holes took grout so freely that a vacuum existed at the surface and a funnel was installed on the header at the hole. Ordinary concrete sand was shoveled into this funnel where small water jets aided its flow. Knowing the rate at which the grout was being pumped, it was possible for the operator to proportion the mix by adding sand at a fixed rate. In this manner, the sand was handled quite satisfactorily, and a large quantity was placed in the two holes. However, the sudden and peculiar manner in which the holes refused raised some question as to the wisdom of accepting this material, and it was discontinued. Subsequent redrilling and regrouting of the holes that had refused to take more of the sand grout further confirmed the belief that it was unsuitable, as it was found that it was possible to pump a large quantity of the cement rock flour grout into the same seam that had been previously plugged with the sand mixture. Cores of the sand grout that were later removed from nearby holes indicated a strong tendency toward segregation, the material being lean and crumbly and of a generally poor quality. No doubt segregation could be reduced by the use of find sand. In general, the writer believes it unwise to use sand for the primary grouting of seams in a foundation that will be subjected to more than a moderate head.

Conclusion

The treatment of every foundation produces new and unforeseen problems to which judgment and experience offer the best solution. By reason of its

nature, the work does not lend itself to control by rigid specifications and, once the broad plan of approach has been determined, the responsibility for the changes that will be necessary in order to cope with the various emergencies that will arise is best placed upon the field forces. It is seldom that the quality of the work can be permitted to suffer from the delays incidental to consultations and special studies, and the changes are usually in the nature of emergencies. For this reason, it is well to choose inspectors possessing initiative and judgment for directing the work.

The uncertain character of the work makes it impractical to estimate the cost of foundation treatment in advance with any assurance that the values obtained will be better than a guess. It is true that, although subject to wide variation, unit costs may be approximated, but it is difficult to approximate the total quantity of materials that will be required. Having no basis of comparison, therefore, it is not easy to judge whether the work is being performed economically. However, by keeping the object of the work in mind constantly and by so regulating the pumping rates and fixing the refusal pressures that the least material commensurate with the maintenance of the desired standard of quality is used, appreciable economies may be effected. In practice, this means that a study must be made to determine the characteristics of every hole or group that is to be grouted and that pressures, pumping rates, and water-cement ratios must be varied to suit the individual peculiarities of each.

Modern methods of foundation treatment leave much to be desired from the standpoint of both effectiveness and cost, and the obscure nature of the work often causes it to receive less attention than its expense warrants. It is a subject in the interest of which much study and experimentation could profitably be undertaken, as it is probable that, as available sites are utilized, it will be found frequently that the foundations present greater problems than the structures. The discovery of suitable cheap materials, inert or otherwise, and in a form to be mixed with cement or to be used alone, would be a great step forward in the field of foundation treatment. The extended use of chemicals for grouting or for varying the properties of conventional grout mixtures also has interesting possibilities for obtaining economies through better control of the limits of the work. The manufacturers of drilling equipment have made noteworthy improvements in their products and in the development of new aids to drilling in recent years, and this progress is reflected in the lower drilling costs that prevail today. In the future, the major economies that will be effected in the field of foundation treatment will probably result from the development of cheaper materials for grouting.

Acknowledgments

The writer wishes to acknowledge gratefully the valuable help received from both the construction and engineering forces of the TVA in performing this work with the exactitude and attention to detail that was necessary. Credit is especially due to G. A. Carlson and J. C. Evans, Jun. Am. Soc. C. E., for their assistance in directing the work, as well as for their aid in preparing this paper.

FOUNDATION EXPLORATION AND GEOLOGIC STUDIES AT GUNTERSVILLE DAM

By Robert M. Ross,[2] Esq.

Synopsis

The construction of a dam in the vicinity of Guntersville, Ala. (see Fig. 22), has been under consideration since 1914. Geologic investigations of several proposed sites were made in that year by the U. S. Army Engineers, and the work was continued in 1933. The first investigations by the Tennessee Valley Authority (TVA) were begun in the fall of 1934. A detailed geologic map of

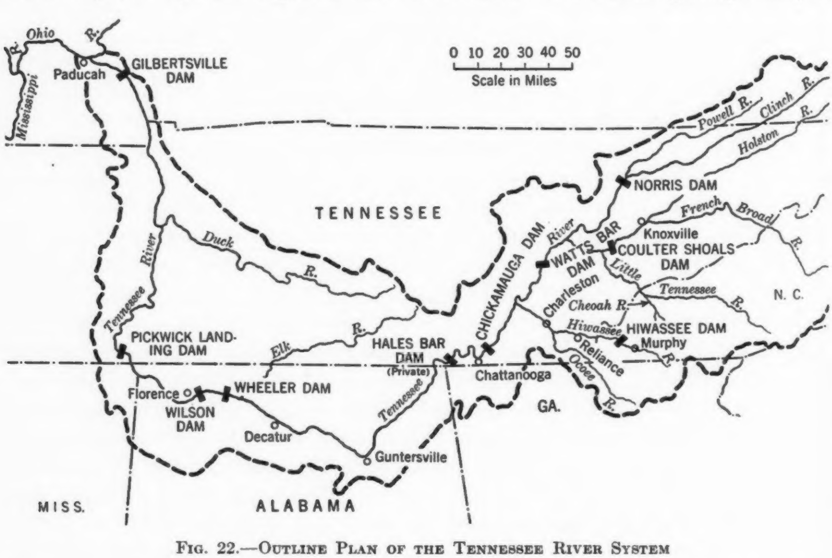


Fig. 22.—Outline Plan of the Tennessee River System

the region was made, stratigraphic studies of the various formations involved were conducted, numerous geologic sections were measured and described, and considerable structural data were accumulated. Eight dam sites were examined. Each was subjected to an intensive study, which was greatly facilitated by the diamond-drill cores previously obtained by the Army Engineers. This paper is a brief summary of the geological studies upon which the final design was based.

Regional Geology

The rocks in the Guntersville area are all sedimentary, and all belong to the Paleozoic system. During past geologic time they were subjected to power-

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ful compressive stresses from the southeast which folded them into a great arch, known as the Sequatchie anticline. The arching of the strata caused them to crack and to become highly susceptible to the forces of weathering so that the anticline became eroded until today it is expressed topographically as a deep valley. The Sequatchie Valley is 4 or 5 miles wide at Guntersville; it continues south about 20 miles and extends far into the State of Tennessee, to the north.

Because of the folding and the erosion which followed it, the various formations present outcrop in long, parallel belts. The middle part of the

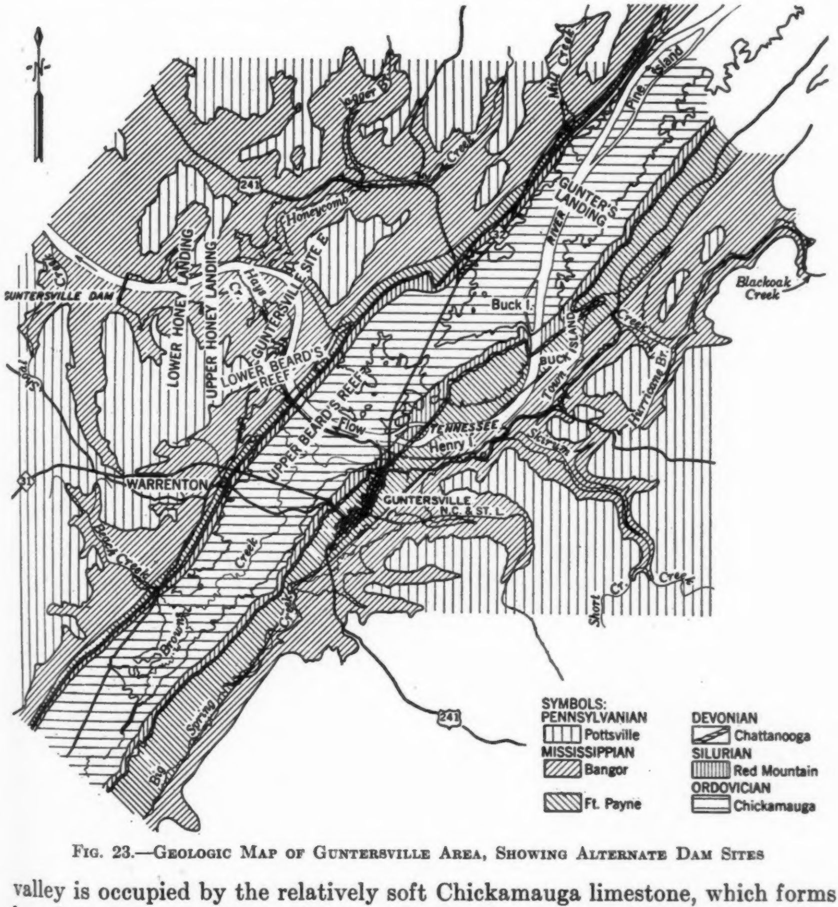

FIG. 23.—GEOLOGIC MAP OF GUNTERSVILLE AREA, SHOWING ALTERNATE DAM SITES

valley is occupied by the relatively soft Chickamauga limestone, which forms low, flat areas. On either side of the Chickamauga limestone, forming a pair of straight, much dissected, subordinate ridges, are the Red Mountain formation, which is mainly limestone and calcareous shale, the Chattanooga black shale, and the Fort Payne chert (see Fig. 23). The soluble Bangor limestone occupies the valleys between the ridges and the bordering escarpments and composes the lower part of the escarpments. The resistant Pottsville formation caps the high plateau areas which lie on either side of the valley.

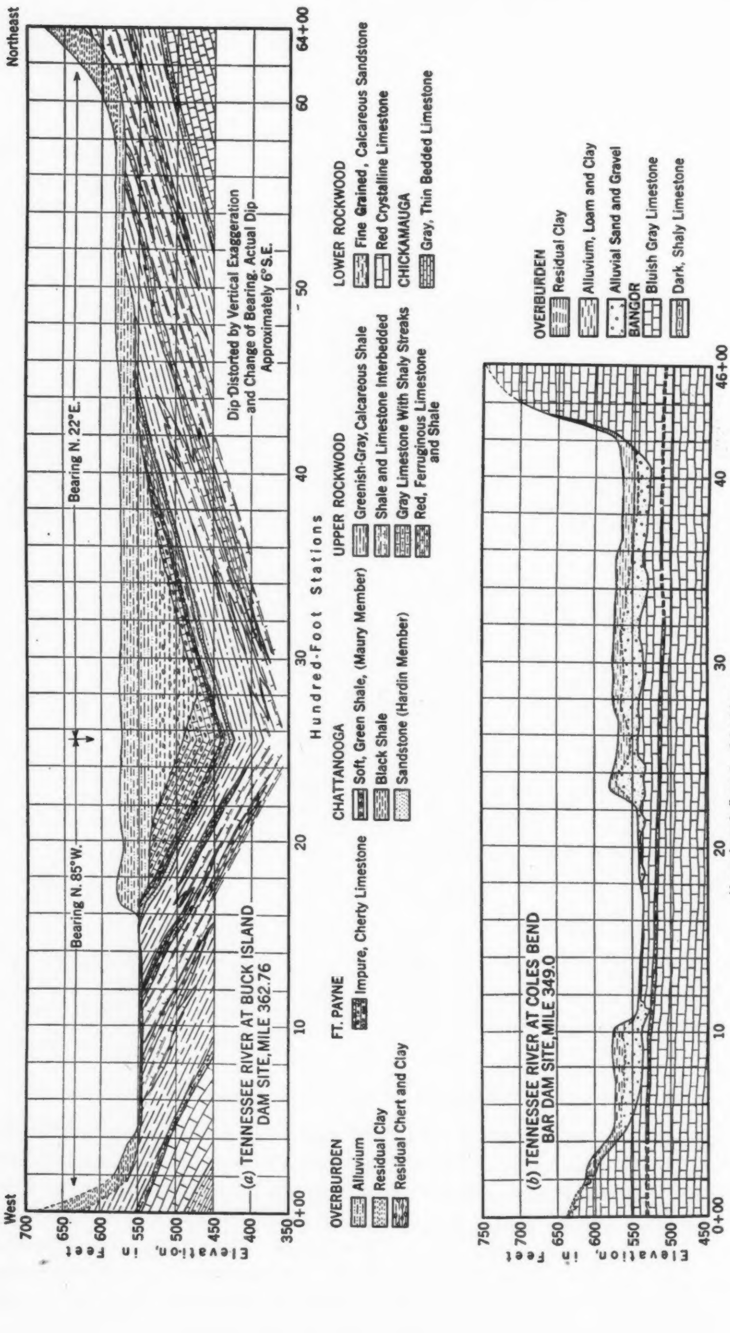

FIG. 24.—GEOLOGIC SECTIONS OF COLES BEND BAR (GUNTERSVILLE DAM) AND BUCK ISLAND SITES (HORIZONTAL SCALE, 1 IN. = 200 FT; VERTICAL SCALE, 1 IN. = 50 FT; $VE = 4$)

Selection of the Site

Six of the eight original sites were eliminated from consideration after the preliminary studies. They were all in soluble limestone formations, and at each site the foundation rock had suffered extensive solution and contained numerous cavities. This condition made the sites very unsuitable for the erection of a dam.

The two remaining sites were the Buck Island site, about 4 miles upstream from the town of Guntersville, and the Coles Bend Bar site, 9 miles downstream. A thorough diamond-core drilling program was begun at both sites in July, 1935. The work at Buck Island was completed in September and that at Coles Bend Bar in the following May (see Fig. 24).

The Buck Island site was originally located in the Chickamauga limestone. This was too cavernous to be acceptable so the site was transferred 0.7 mile downstream, where the more resistant, ridge-forming rocks crossed the river in narrow belts. It was hoped that the Red Mountain formation, the Chattanooga black shale, and the Fort Payne chert might provide a suitable foundation. The first two came up to expectations, but the Fort Payne, although it forms a relatively sound, resistant foundation for both Wilson and Wheeler dams, was deeply weathered at Buck Island and utterly unsuitable. At some points it had deteriorated to a mass of loose chert and clay to depths of 140 ft below the surface. The satisfactory formations were so narrow that it was impracticable for them to support the entire dam. In contrast, the rock at Coles Bend Bar, although limestone, was found to be unusually sound. These conditions, together with other factors, such as the added cost of dredging to the Buck Island site, resulted in selecting the Coles Bend Bar site.

The Guntersville Dam Site

The axis of Guntersville Dam crosses the Tennessee River at right angles, is 3,985 ft long, and has a bearing of North 18° East. The central part is of the concrete gravity type and consists of a navigation lock, spillway, and power house. On either side of the river, flanking the masonry section, are earth embankments which extend across the flood plains to the abutments.

Topography.—The north flood plain is about 600 ft wide and has an elevation of from 29 to 33 ft above low water. The south flood plain is about 1,800 ft wide and has an elevation of 21 to 37 ft above low water. Both abutments are at the bases of steep hills which rise 500 ft or more above the river.

Geology.—Throughout most of the flood plain the alluvium is roughly divisible into an upper part consisting of loam and clay and a lower part consisting of sand, or sand and gravel. The loam and clay, according to results obtained from earth auger borings, averages slightly more than 10 ft in thickness. It is relatively impervious. The underlying sand and gravel is usually between 20 and 25 ft thick and is often very permeable, especially just above bedrock. In order to intercept the flow through these permeable areas and prevent possible leakage, it was necessary to drive steel sheet piling to bedrock along the entire lengths of both earth embankments. At the abutments, the alluvial material gives place to a heavy, brownish-red, residual clay which is extremely stiff and impervious.

The site is located in the middle portion of the Bangor limestone. The formation rises about 350 ft above river level and probably extends downward as much as 300 ft. The foundation rock is uniformly hard, massively bedded, rather dark bluish-gray, crystalline limestone, except for a relatively thin stratum of shale and shaly limestone. The shaly stratum has an average thickness of about 3 ft. It is of wide horizontal extent, there being only two small areas in the vicinity of the site where it appears to be absent. One of these is at the left bank of the river and the other extends beneath the flood plain from the left abutment.

The bed is divisible, characteristically, into three members. The upper and lower are black or dark gray, thin-bedded, considerably weathered, carbonaceous shale. The middle member is dark gray, compact, shaly limestone. It is likely that, originally, the entire stratum was shaly limestone, but the upper and lower portions, being next to the bedding planes and more accessible to percolating water than the middle, had part of their calcareous substance leached out and the residuum compacted and converted into shale. Chemical analyses reveal the bed to be much lower in carbonates, and consequently much less soluble, than the limestone above and below it.

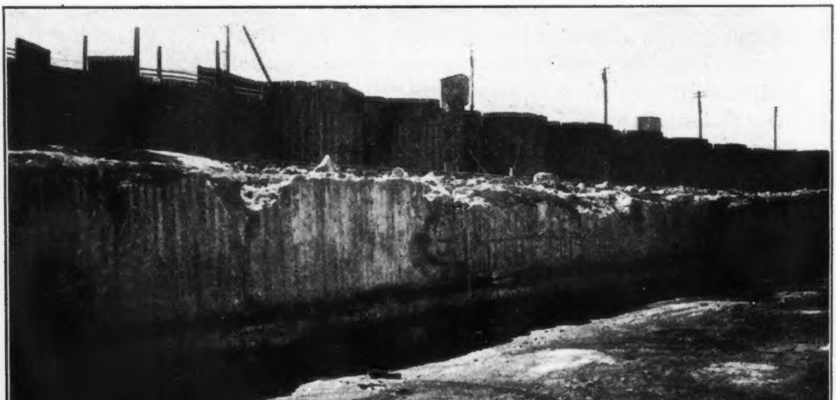

FIG. 25.—SHALY STRATUM EXPOSED IN LOCK SECTION, GUNTERSVILLE DAM

The shaly stratum has had a profound influence on the foundation rock and is probably responsible for its unusually good condition (see Fig. 25). Since it is relatively impervious, it has served to shield the strata below it from percolating waters and hence to prevent solution. Because of its presence, the deep seams and cavities which ordinarily occur in the Bangor limestone, beneath the

river, are very largely absent. Along the axis the stratum is overlain by from 2 to 30 ft of limestone. In some areas of this rock, solution is extensive, whereas below the shaly bed the rock is remarkably sound.

The effectiveness of the bed in protecting the rock below it is demonstrated graphically by the logs of the diamond-drill holes at the dam site. Altogether,

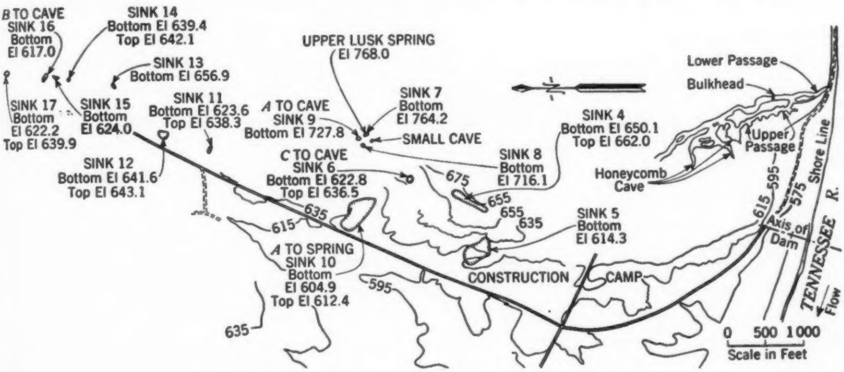

FIG. 26.—HONEYCOMB CAVE AND RELATED SINKHOLES

186 holes (110 of which penetrated the horizon of the shale) were drilled under the direction of the TVA; 264 seams were present, and of this number 245 were above the level of the shaly bed. Of the 19 that occurred below it, 15 were in areas where the shaly bed was indistinguishable and only 4 were below a recognizable shale.

The bedding is practically horizontal. There is a dip to the south along the axis of about one-fourth degree. This is not the true dip since the axis is not parallel to the strike. The strike is approximately North 35° East and the regional dip is something less than 1° Southeast, or upstream, although there are slight irregular undulations in the rock which cause local variations of 1° or 2°.

Occasional slickensides and small calcite veins, indicative of pressure from the southeast in times past, may be seen. These are too slight to have any practical significance. There are three distinguishable joint sets. Two are formed of shear joints, produced by compressional stresses. One of these has an approximate trend of from North 10° West to North 25° West; the other trends between North 70° West and North 85° West. The former is the dominant set. The caves and the solution channels in the vicinity usually follow it, having occasional cross branches developed along the second set. The third set is composed of strike joints which vary considerably but nearly all lie between North 10° East and North 30° East.

Condition of the Foundation.—The effects of solution constituted the only serious foundation problem at the dam. The limestone contained an average of more than 90% carbonates and sometimes more than 98%. As a result, cavities and enlarged joints were numerous in some areas, although the shaly stratum, which may be considered the lower limit of appreciable solution, confined them largely to the surface beds (see Fig. 26).

With the exception of the shale, the foundation rock is hard and resistant to stress. Compression tests on a number of samples revealed an average crushing strength greater than 23,000 lb per sq in. Samples of the weathered shale were far weaker, averaging only 155 lb per sq in.; but it must be remembered that the shale tends to soften and crumble on exposure to the air, and the samples tested had been so exposed for several hours at least. The shaly stratum is sufficiently indurated not to soften under the action of water and, when overlain by the limestone, appears easily capable of supporting the weight of the dam.

Under certain circumstances, sliding of the foundation might occur along the shale, but at the dam site such a tremendous volume of rock would have to be moved and lifted, because of the low upstream dip, that it is out of the question. There are no important structural complications and no deep channeling in the foundation.

The block diagram, Fig. 27, is intended to convey a general idea of conditions in the foundation rock. It was impossible to illustrate conditions properly and preserve strict dimensional accuracy; therefore, the sizes of the geologic structures in Fig. 27 have been exaggerated about four times. The general level of bedrock, in the lock section, was rather constant, but there were areas of extensive cavities. In some places solution had progressed along joints

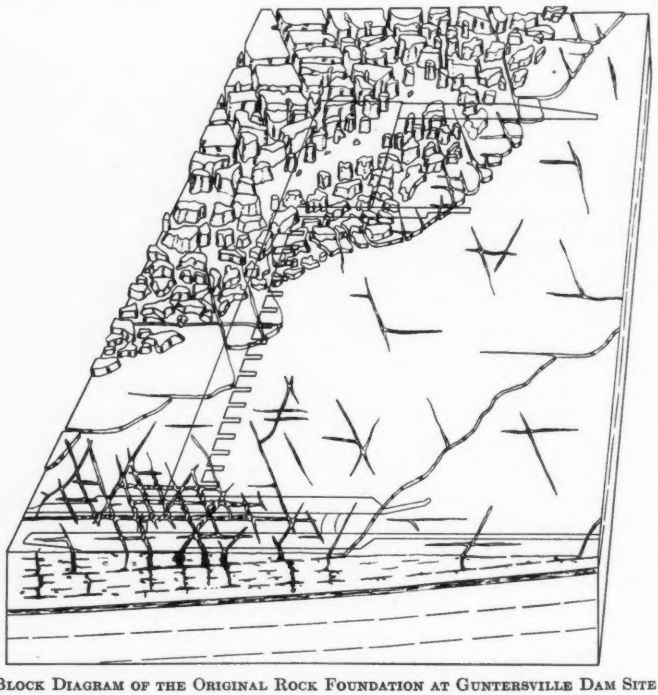

FIG. 27.—BLOCK DIAGRAM OF THE ORIGINAL ROCK FOUNDATION AT GUNTERSVILLE DAM SITE ($VE = 4$)

and bedding planes until a system of crisscrossing, vertical and horizontal openings, was produced. Most of the vertical seams were developed along joints. Such seams ran in more or less straight lines, cutting across each other at high angles, so that the rock was sometimes divided into numerous rhombic

blocks. These were often less than 4 ft in diameter, where solution was far advanced. The horizontal seams developed along the bedding planes. They intersected the vertical seams more or less at right angles and gave rise to a zigzag, step-like system of cavities which often exceeded 5 or 6 in. in thickness and sometimes attained thicknesses of 3 or 4 ft. They were most abundant near the surface and diminished rapidly with depth. Most of them occurred in the upper 8 ft, and there was a definite stratum of highly dissolved rock between 2 and 6 ft below the surface.

Because of the advanced solution in the rock overlying the shaly stratum, the lock foundation was excavated to the surface of the underlying limestone, which was very sound and unweathered. Farther south, in the spillway section, the rock became very smooth and free from seams. There were relatively few open joints, even at the surface, and those that did occur were not very deep.

FIG. 28.—ERODED STRATUM IN POWER-HOUSE SECTION, GUNTERSVILLE DAM

The dip is so low that the upper part of the foundation throughout most of the section was composed of a single stratum. In the southeastern part a thin, overlying bed about 2 ft thick appeared. A third occurred above the other two, still farther to the southeast, where the strata were lowest. It was from 4 to 6 ft thick and had suffered such erosion that it was reduced almost everywhere to discontinuous, isolated blocks and boulders, surrounded by alluvium.

The same three beds continued on into the power-house section. The highest, consisting of a multitude of eroded remnants, covered more than half the section, whereas the lower ones outcropped in the northwestern part (see Fig. 28). The two upper beds were removed, and the lowest provided an unusually good foundation, notwithstanding the fact that its surface was 18 or 20 ft above the shaly stratum.

Thorough core drilling was done throughout the entire masonry section, both for exploration and for grouting and drainage. All the holes were examined carefully and logged. A number of 36-in. holes were drilled, thus permitting a careful visual inspection of the foundation rock to depths of 30 ft or more. After excavation was completed, a detailed map of all joints in the rock was made so that every path for potential leakage would be known in the event that further foundation treatment should ever become necessary.

Diamond core drilling indicated that the rock beneath the north embankment was relatively free from seams, and this conclusion was confirmed subsequently by the holes bored for grouting. The rock surface was smooth except for a few irregularities near the abutment where several residual boulders were present.

The south embankment foundation was much less satisfactory. Ultimately, pits had to be opened over its entire length to permit treatment of the rock. The northern half was underlain by the same greatly eroded bed which was present in the power-house section. Just below it, however, the foundation was found to be unusually sound.

Several extensive, deep cavities occurred under the southern part of the embankment. These cavernous areas were from 27 to 138 ft long, along the axis, and sometimes went as deep as the shaly horizon, more than 25 ft beneath the top of the bedrock. They appeared to continue upstream and downstream for considerable distances and were always partly filled with sand and gravel. The cavity in Fig. 29 was much more extensive than indicated by the shaded area, which shows only the part that was actually excavated and explored. This cavity was cut off by a concrete wall and iron piles. The cavities were ordinarily between 3 and 6 ft thick, but the maximum was nearly 12 ft. Since grouting was considered undesirable in openings of this size, they were excavated and blocked with concrete bulkheads.

The rock in the north abutment was very much decomposed. An extremely widespread system of cavities, with bottom elevations 20 or 25 ft below flood-plain level, existed (see Fig. 30). Many of these cavities were more than 20 ft thick. The hill which forms the abutment is mantled with a reddish-brown, stiff, highly impervious, residual clay, and nearly all of the cavities were filled with this material.

Intensive drilling disclosed the presence of a comparatively solid rock projection in the abutment about 50 ft upstream from the axis. In order to intercept the cavities, a trench was excavated to sound rock, and a concrete cutoff wall built from the axis to the projection. The abutment was later grouted thoroughly.

The south abutment is very broad, steep, and solid. The rock is unusually sound, although its surface is cut by numerous shallow, vertical fissures, and several thin horizontal seams are developed along bedding planes. Ordinarily, these were filled with heavy, residual clay so that leakage appeared unlikely. Nevertheless, the rock was explored carefully with drill holes and grouted.

Honeycomb Cave.—Of all the caves in the vicinity of the dam, only one was so situated that it might cause any trouble. This cave, known as Honeycomb Cave (see Fig. 26), is in the plateau remnant which forms the right abutment. The mouth of the cave is about 2,000 ft upstream from the dam

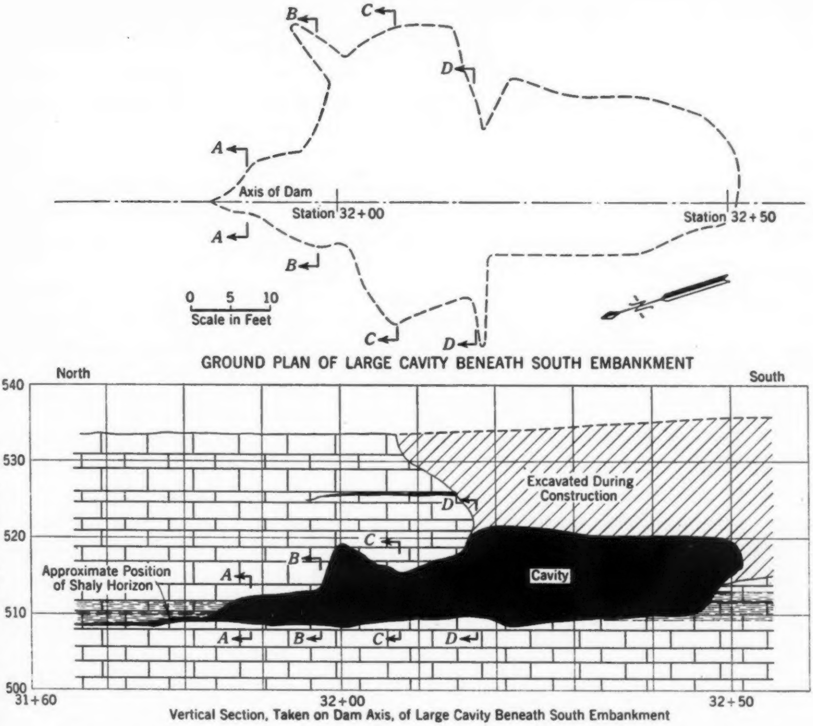

FIG. 29.—LARGE CAVITY IN FOUNDATION ROCK BENEATH THE SOUTH EMBANKMENT

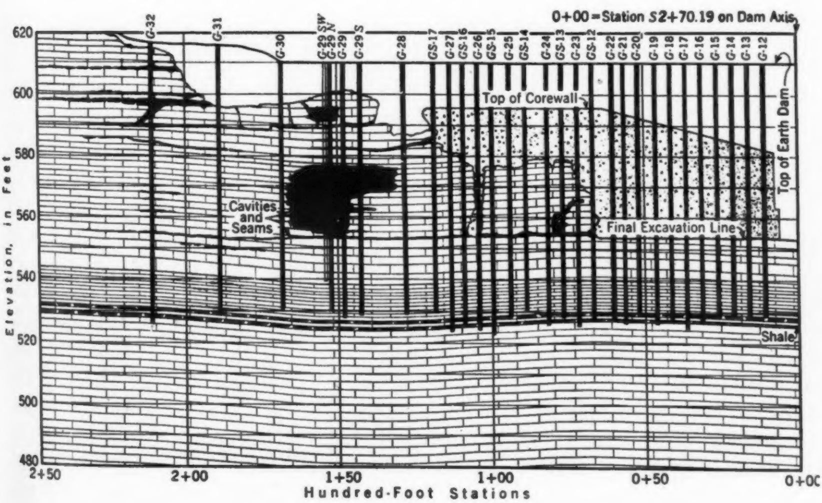

FIG. 30.—CAVITIES AND LOCATION OF DRILL HOLES; VIEW FACING UPSTREAM ALONG AXIS OF NORTH ABUTMENT, GUNTERSVILLE DAM

site, and many of its inlets occupy a valley to the west, which empties into the river downstream, so that there is a conduit around the abutment. An intensive examination of the cave and its inlets was made during the summer and fall of 1935. It was explored for a distance of about 2,500 ft, and a detailed map was prepared. After the passages of the cave had been traced as far as was practicable, a study of the inlets was made from the surface of the ground.

The main passage is 15 or 20 ft high and between 20 and 40 ft wide. Its floor is from 16 to 39 ft lower than normal pool elevation. Seventeen sinkholes lie on the slopes or along the western base of the hill which the cave occupies. Most of them represent inlets.

The connection between the sinkholes and the cave was proved by the use of fluorescein, a powerful organic dye which has a characteristic green color and is easily detectable in concentrations as low as 1 part to 100,000,000 parts of water. A solution of the dye was placed in a sinkhole and washed down, after which samples were examined hourly from every available point of exit. After thirty-eight hours the stream flowing through the cave showed color, and this color was maintained for several days, until the entire amount of dye had passed. Subsequently, the same results were obtained with two other sinkholes, one of which was approximately 2 miles from the mouth of the cave.

Since the lowest of the sinkholes was slightly above maximum pool level, it was impossible for leakage to occur directly through any of them. There was a strong possibility, however, that leaks might develop through openings in the soil and rock beneath them. To prevent this condition, the cave was blocked by means of a concrete bulkhead in April, 1937. The stream in the cave was impounded by the bulkhead and rose to an elevation of about 600 ft, which is 5 ft higher than maximum pool. Since the water first rose, it has never been lower than about 590 ft and has stood above 595 ft for months at a time without any leakage into the valley below the dam. Consequently, the bulkhead has proved not only that it is an effective seal but also that no water could escape from Guntersville Lake through the cave, even if there were no bulkhead.

Summary

The rocks near the Guntersville Dam are all sedimentary and, at the level of the river, are mainly limestones and shales. The Guntersville Dam site was selected, in preference to seven other proposed sites, because of the superior condition of the foundation rock and the favorable location of the site. The central part of the dam is a concrete gravity structure and is flanked by earth embankments on either side. The entire site is underlain by the Bangor limestone, which is uniformly hard and massively bedded except for a 3-ft stratum of shaly material. Solution had advanced to a considerable degree in certain parts of the rock above this stratum but was negligible below it, the shale having served to shield the underlying rock from percolating water. The rock was free from structural complications and the effects of solution constituted the only serious foundation problem. These defects were rectified by excavating, grouting, and the construction of concrete walls and bulkheads. A large cave near the right abutment formed a conduit through which it was feared leakage might occur. After thorough exploration, the cave was blocked effeccively with a concrete bulkhead.

FOUNDATION CONDITIONS AND TREATMENT AT GUNTERSVILLE DAM

By Verne Gongwer,[3] M. Am. Soc. C. E.

Synopsis

Foundation conditions at Guntersville Dam presented interesting engineering and construction problems, the solutions of which are thought to be unique. Since techniques for foundation treatment were developed progressively to suit the varying conditions encountered, the different parts of the foundations are described in the order in which they were uncovered. Ordinary procedures, conditions, and methods are described only briefly to permit a more complete description of extraordinary conditions and ground-water behavior after the filling of the reservoir.

General Features of the Project

Guntersville Dam consists of a reinforced concrete spillway structure 856.0 ft long in the river channel. On the right or north bank there is a navigation lock, with a chamber 60 ft by 360 ft; and on the left or south bank is the

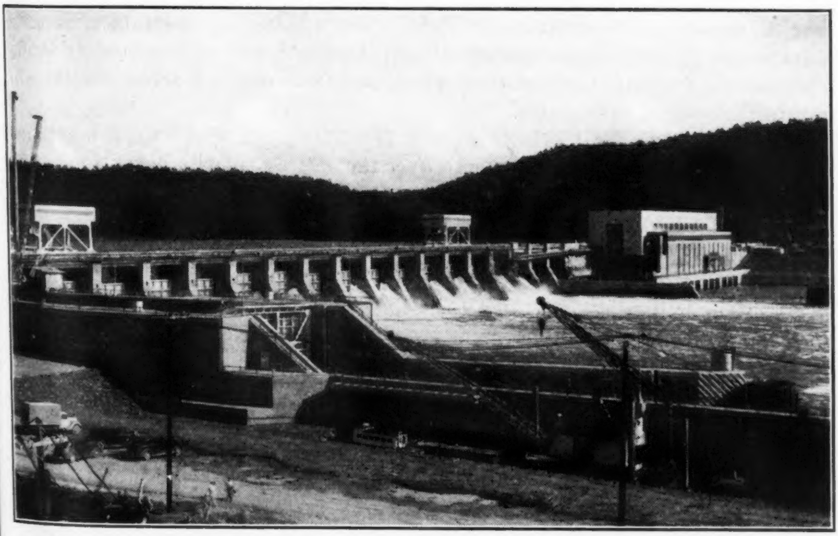

Fig. 31.—Guntersville Dam, Showing Lock, Spillway, Power House, and Switchyard

power house with three 24,300-kw generating units, space being provided for a fourth unit when it is required (Fig. 31).

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The rolled earth-fill ends of the dam abut against limestone bluffs about 4,000 ft apart at pool level. The left abutment is quite precipitous and regular from the rock floor of the valley to pool level, whereas the right bluff, from the rock floor to pool level, consists of overhanging limestone strata, rendered exceedingly ragged by the processes of solution, remarkably eroded, and overlain generally with clay 4 to 6 ft deep. The lock, spillway, and power-house structures lie nearer to the right bluff than the left, and the remainder of the dam consists of rolled earth dikes, approximately 800 ft long on the north and 1,600 ft long on the south, with upstream and downstream slopes of 1 on 3, and heights of from 35 ft to 45 ft.

In general design, Guntersville Dam is somewhat similar to Chickamauga Dam and Pickwick Landing Dam, but the foundation conditions are dissimilar to a large extent.

From a construction viewpoint, the site of the dam is in a deep deposit of Bangor limestone, in which the river has eroded a wide, comparatively shallow canyon, with a fairly level rock floor. Upon this floor silt and alluvium average about 35 ft deep. The materials are arranged in general with quite permeable gravel, from 0 to 5 ft thick, lying directly upon the rock, and fine gravel, coarse sand, fine sand, sandy silt, and clay (in the order named) from the bottom to the top of the flood plain. The stream bed, approximately 1,100 ft wide at the dam site, has been swept clean down to the rock in some places.

The rock floor has an inclination of about 1% to the south and is practically level upstream and downstream. The limestone is bedded in strata of various thicknesses dipping approximately 1% to the south and approximately 0.5% upstream or easterly, thus causing certain strata to outcrop across the stream within the several cofferdams.

Roughly, 15 to 25 ft below the rock floor along the dam axis, a comparatively thin double-shale seam extends over the entire site, with exception of one or two small areas. The shale seam, generally, is in two members from 6 to 12 in. thick with a limestone member between, ranging from 2 ft thick to 0, where the shale members blend into one, and even disappear over the small areas mentioned.

Preliminary borings (by the U. S. Engineer Department), generally 100 to 250 ft apart, in several ranges, and later borings, indicated general conditions and indicated the selection of this particular site as compared with others near the head of Wheeler Pool. The location of Wheeler Dam (Fig. 22), and the approximate pool elevation, had previously been fixed by the construction of Wheeler navigation lock by the U. S. Engineers.

The preliminary borings, however, did not reveal the peculiar detail features of the rock, which later presented several construction difficulties. During early construction it appeared that difficulty might be encountered for the entire length of the dam. However, as the work progressed advantage could be taken of certain favorable conditions that resulted in ultimate costs well within the preliminary estimates. Originally, only the intakes for a future power house were contemplated and approved for construction, no generating units being authorized. Circular-cell, steel sheet piling cofferdams were adopted to exclude

a maximum height of water of approximately 40 ft above the top of rock. This river stage should be expected about every three years, but was nearly reached during three of the four ensuing winters.

Permeable gravel samples from certain test pits, the comparatively high specific capacity of some of the pits under pumping tests, and the fairly smooth rock floor indicated by the preliminary borings, resulted in an early decision to drive a steel sheet pile cutoff to rock across the entire width of the flood plains.

Steel Pile Cutoff Under North Embankment

From the lock wall nearly to the north abutment, the sheet piling was driven to a very uniform rock surface. However, near the north abutment the piling (see Fig. 32) ran up over several large residual boulders embedded in residual clay, with a thin edge of the pervious gravel layer beneath. These boulders were removed and the piles were trimmed and set down upon the smooth rock floor beneath. The backfilling of this excavation was compacted with air tampers, thus ensuring practically ideal compaction. The sealing of the piles to the rock floor was otherwise considered satisfactory, and the seal to the rock escarpment was obtained by means of a concrete cutoff wall.

Lock Foundation

The lock cofferdam was unwatered and cleared of mud, gravel, and loose boulders in moderate number and size. The hydraulic dredge was only partly successful because of boulders and eroded rock surface. Cleaning of an area for a pump sump revealed enlarged joint cracks at approximately right angles, dividing the top stratum of rock into nearly rectangular blocks (Fig. 33). As

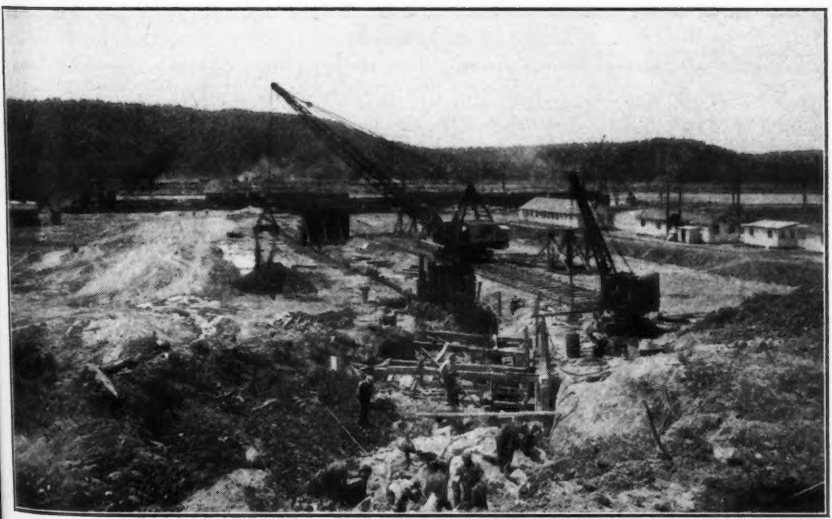


Fig. 32.—Steel Pile Cutoff Under North Embankment, Showing Trench for Cutoff Wall

the sump excavation progressed, the bedding planes and joint cracks were found to be peculiarly enlarged and distorted by solution (that is "solutionized"), the joint cracks usually being narrow at the top, widening downward,

Fig. 33.—Joints and Solution Channels in the Top Stratum of Rock in the Lock Cofferdam

and staggered somewhat in the different strata. This condition continued downward to the general shale seam.

Previously probings were made from a barge in an attempt to explain a few irregularities of the rock floor indicated by the more widely spaced preliminary drill holes. The probings revealed what were now identified as the joint cracks enlarged by solution. The probing had defined many of these open seams over an area from 50 to 75 ft wide, in a band or path 50 to 75 ft wide running slightly diagonal with the lock and out beneath the upstream outer corner of the cofferdam. At the latter place leaks of considerable magnitude developed early (see Fig. 34) since many "solutionized" bedding planes were filled only partly with quite permeable material. "Solutionizing" near the upper end of the lock had progressed so far that the upper strata had subsided 2 or 3 ft in some places and were absent at other points. The excavating and hauling equipment frequently broke through these undermined layers, resulting in lost time and high repair costs.

Exploration was expedited with wagon drills and small and large shot drills. The 36-in. shot-drill holes, at frequent intervals, were a speedy and especially satisfactory means for examination. It was soon found that in areas where the action of solution was most pronounced, most of the rock above the shale seam was questionable, with solution channels occasionally amounting to small caverns, whereas below the shale the bedding planes were generally very tight and vertical joints few and equally tight.

Due to the slight inclination of the strata, the shale rose to within about 3 ft of the rock surface near the downstream end of the lock. The excavation, starting there, was closely drilled to concrete lines and extended to the bottom

FIG. 34.—LEAKS THROUGH SOLUTION CHANNELS BENEATH UPPER END OF LOCK COFFERDAM

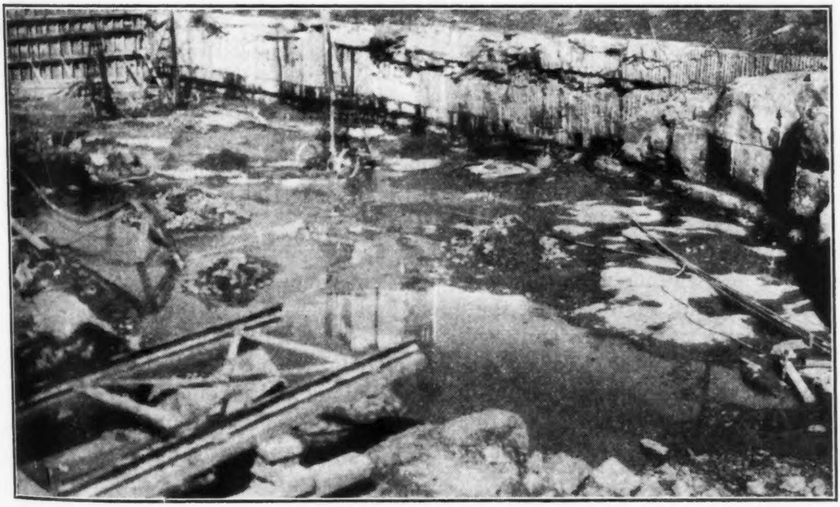


FIG. 35.—LOCK WALL FOUNDATION, SHOWING SHALE SEAM NEAR THE BOTTOM OF THE EXCAVATION

of the shale seam (which can be seen in Fig. 35). As the shale dipped deeper upstream an attempt was made to save a local area of quite sound rock. A fairly heavy blast disposed of the matter by shifting a portion of this rock along the shale about 3 in., as revealed in one of the 36-in. holes. The entire lock excavation, therefore, was extended to the bottom of the shale.

The remaining minor seams in the rock below the lock foundations were treated by drilling low-pressure holes on 20-ft by 17-ft centers and grouting at 30 lb per sq in. before placing concrete. A curtain of high-pressure holes was then drilled at 20-ft centers across the upper miter sill to connect with a similar curtain under the spillway and under the north dike. Pipes were extended through the concrete and these holes were later grouted at a pressure of 60 lb per sq in. Seepage into the excavation from below the shale was negligible, even before grouting, and little grout was taken by either low-pressure or high-pressure holes. A sound area of rock above the shale landward of the lock furnished a tight foundation and a connection for the steel pile cutoff of the north dike.

During operations in the lock cofferdam, several serious leaks or blows occurred, filling the cofferdam several times and losing a total of approximately six weeks' progress. Temporary closures were made several times from the river side by means of many barge loads of sand bags, gravel, and baled hay, protected from river scour by a row of timber cribs. After one such blow had been stopped a tier of blocks between vertical solution channels and parallel to the cofferdam was removed and the space filled with concrete. (In Fig. 36, note the open horizontal seam extending under the cofferdam. Convenient vertical seams form the sides of the trench.) This horizontal solution channel reached the full distance beneath the cofferdam. In driving, the piling of the cells had seated firmly upon the level upper rock stratum, causing the tops of the piles to appear very uniform and satisfactory. At several points "chimneys" in the rock sucked in the cell filling as the blows began and threatened to collapse the cells. Failure of the cells was prevented by prompt plugging with sand bags and baled hay.

Attempts to grout the seams under the upper end of the cofferdam through wagon-drill holes immediately inside the cells were defeated by flowing water. Blanketing these particular leaks on the river side of the cofferdam by means of hay, gravel, sand bags, etc., met with no material or permanent results. Outside blanketing was unduly expensive, since it was impossible to locate all of the numerous entrances to the seams, and since the work required a large force of men and much floating equipment of high operating cost. As the lock walls were built high enough, the upper and lower ends of the lock were barricaded with timber cribs and needle dam, respectively, to allow the lock floor and other work in the chamber to be completed. Stoppage of leaks by grouting the open seams through a line of drill holes along and through the center line of the cells with the cofferdam full of water was discussed; but the necessary delay was deemed too great, and the results too problematical.

Spillway Foundation

After the second cofferdam had been started out into the river, it was decided to drill through the center of the cells on 10-ft centers and grout all seams in the rock and the gravel filling of seams under and between residual boulders. A second row of holes was added, staggered with the first, except for the downstream arm of the cofferdam. Here conditions appeared to be better, and 20-ft spacing was adopted. The holes were drilled through the shale a few

feet. The ragged profile of the tops of the piles in certain locations, as seen in Fig. 32, indicated groups of residual boulders at the bottom of the piling. Smooth appearing piles were found frequently over badly undermined layers.

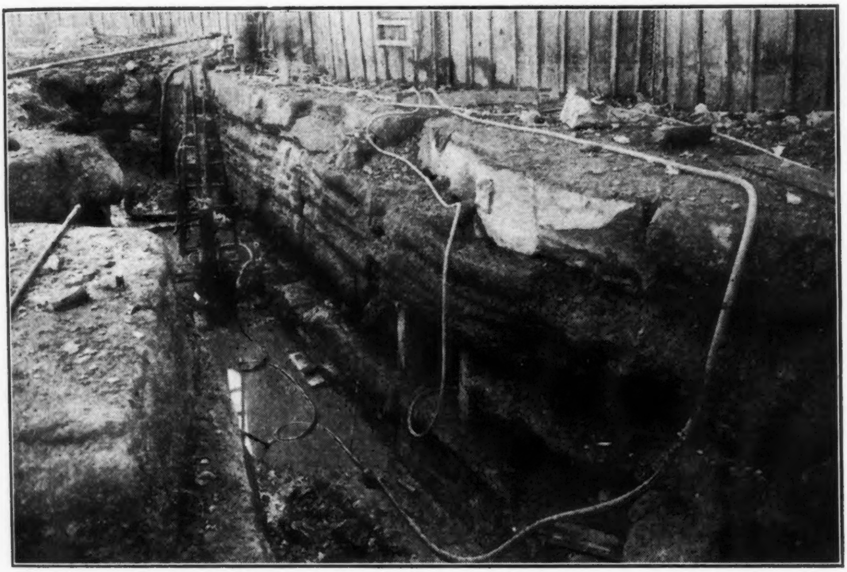


FIG. 36.—TRENCH FOR SEALING HORIZONTAL SOLUTION CHANNELS THAT HAS PERMITTED COFFERDAMS TO FILL

The grout holes were located along the center line of the cells in order to render the grouting most effective by insuring that, whichever way the grout traveled from the hole, the pervious seam-filling material or open seams would be solidified for a distance equal at least to one half the width of the cells. Outside of the cells the grout could usually escape upward through vertical "chimneys." Escape of grout either into the cofferdam or into the river could usually be detected by bubbles, froth, or cement color and the grouting was slowed or stopped until it began to stiffen and a sufficient back pressure was indicated (by gages on the grout lines). Special holes were drilled and grouted slowly opposite the "doorways" under the piling caused by boulders.

The resulting total leakage into this cofferdam was negligible, requiring only part-time operation of one 6-in. pump; yet the head on the cofferdam (caused by filling Wheeler Reservoir a short time previously) was constantly equal to (and greater than, during a July flood) the head produced by occasional river rise, which had invariably caused the lock cofferdam to blow. Grouting a few nominal leaks against full cofferdam head (with cement, sand, and sawdust mixtures) met with little success.

As the excavation progressed, it was found that the particular rock strata which gave trouble in cofferdam No. 1 were generally quite sound, but that two or three superimposed strata existed which were extensively affected by

solution. Had the grouting not prevented, this condition obviously would have permitted quite as troublesome leaks and blows as in cofferdam No. 1.

Construction in cofferdam No. 2, due to the resulting dry and favorable conditions, was completed in approximately four months, obviating expensive delays and disruption of the construction schedule. The direct cost of drilling and grouting was substantially less than the various and repeated remedial measures necessary in cofferdam No. 1 Also it was imperative that work be completed in cofferdam No. 2 in this remarkably short interval in order to complete the third cofferdam and divert the river through the spillway before the winter floods.

Detailed exploration of the rock in cofferdam No. 2 revealed that, except for the first spillway bay south of the lock, there was a depth of 16 to 18 ft of sound rock (Fig. 37) above the generally occurring thin shale strata; and the plans were revised to set the spillway on this rock, thus obviating an estimated $300,000 worth of line drilling, rock excavation, and reinforced concrete necessary to carry the piers and also the weirs down to the stratum below the shale. This also saved much valuable time and indirect cost.

When cleaned off, the river-bed rock was practically level and characteristically scoured by river action, with grooves and ripples as seen in Fig. 38. There were a number of tight vertical-joint cracks, and a few which were enlarged by solution ("solutionized") nearly to the shale. These were cleaned thoroughly and sealed with concrete.

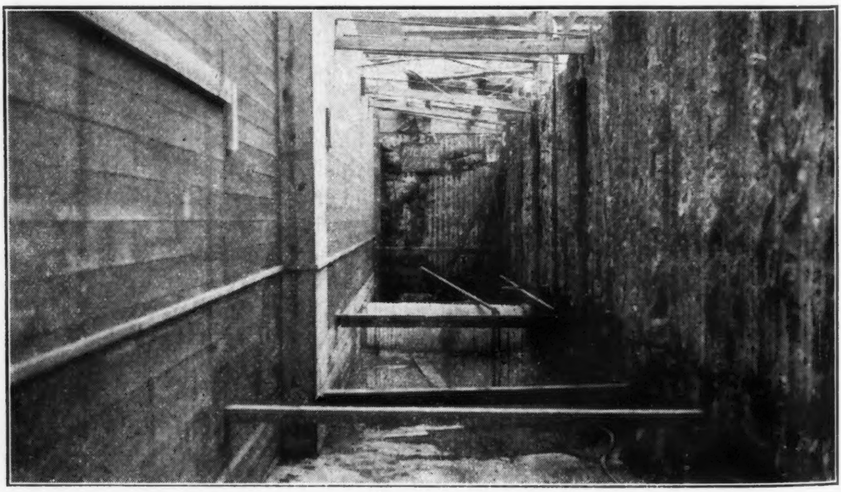


FIG. 37.—EXCAVATION FOR SPILLWAY PIER NO. 2, SHOWING SHALE SEAM AND SOUND ROCK ABOVE

It was also necessary to remove one or two top strata for a short distance to eliminate the extensive partly open horizontal seam which had been the cause of much trouble in cofferdam No. 1. In this seam were found quantities of the characteristically colored sand and clay used in blanketing operations for cofferdam No. 1. As this general seam also ran beneath the upstream arm

of cofferdam No. 2 it had evidently been sealed thoroughly, this time by the grouting operations.

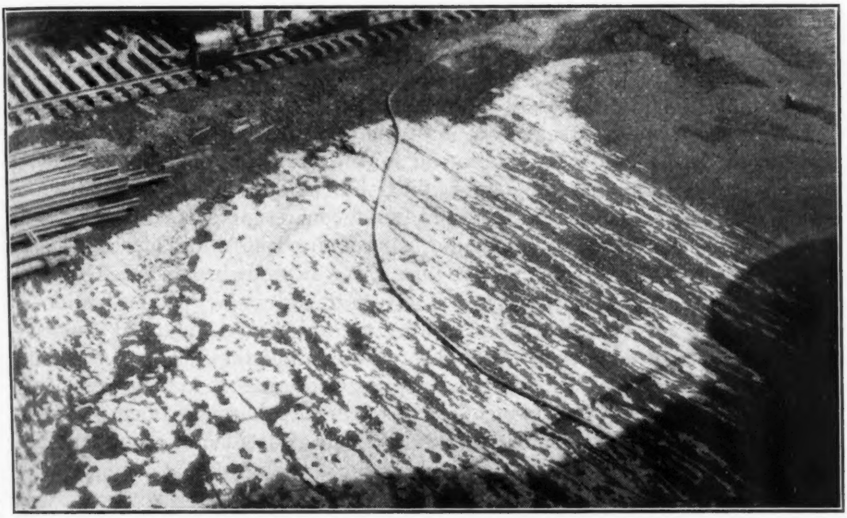


Fig. 38.—Appearance of River Bottom in a Part of Spillway Cofferdam No. 2

As a basis for the decision to save the sound strata above the shale, 36-in. drill holes for inspection were sunk to, and through, the shale at the upstream and downstream ends of each spillway pier and also on many joint cracks. The shale here was found to be relatively hard and intact. Before filling the holes with concrete, baskets of heavy steel reinforcement were inserted. Grout at 30 lb of pressure in holes piercing the shale seams 3 ft horizontally from the vertical rock face at pier No. 2 (see Fig. 37) did not appear at the cut rock face. As a precautionary measure, grouting of wagon-drill holes over the entire area of the spillway on 20-ft squares piercing the shale seams was attempted, but was abandoned as the shale seam was taking insufficient grout to warrant the operation.

As a precaution against erosion or lifting of the river-bed rock strata downstream from the spillway apron, 1.25-in. square dowels were grouted into wagon-drill holes extending several feet below the shale on 5-ft centers each way, over the entire exposed area, about 75 ft wide, between the end of the apron and the cofferdam. Open-joint cracks in this area were cleaned out and filled with concrete. Subsequent soundings indicated that no rock had been displaced by the succeeding heavy winter flood over the completed spillway.

Power–House Foundation

On account of the exceeding dryness of cofferdam No. 2, question was raised as to whether the full grouting technique there used was necessary under the third-stage cofferdam. The driving of a few upstream cells corroborated the expectation that residual boulders, representing the remnants of several

horizontal strata, existed at this point also. An attempt had been made to remove the boulders with a clamshell bucket, but they proved too large and too firmly embedded and interlocked.

One minor drawback connected with grouting of the second cofferdam was already apparent, in that some difficulty was being experienced in redredging the hydraulic cell-filling material from cofferdam No. 2, for use in filling cofferdam No. 3. Some of the grout had escaped upward into the hydraulic filling in certain cells, almost completely impregnating all pervious gravel, pushing aside, compacting, and surrounding all mud or silt contained in the gravel where it was not already completely filling the voids sufficiently to render the gravel impervious to grout. This produced concrete-like masses (Fig. 39), many of which were of sufficient hardness to require drilling and blasting. (All lumps and masses seen in Fig. 39 are pure grout or grout-impregnated gravel.)

However, considering the costly delays and unsatisfactory protection of the blanketing around the first cofferdam, the rapid and economical construction in the second cofferdam which was grouted, and the fact that the third cofferdam must survive two winter and spring flood seasons against somewhat higher heads than the other two cofferdams (the maximum head being approximately 45 ft above the top of river-bed rock), it was decided to grout the third cofferdam in a similar manner.

FIG. 39.—CELL FILLING IMPREGNATED BY CEMENT GROUT ESCAPING FROM BELOW

Since the drilling and grouting had lagged somewhat, in order that results of driving of some of the cells could be observed first, it was finally decided to attempt unwatering while there still remained a portion of the lower arm, nearest the river bank, which had not been grouted. Upon unwatering, the cofferdam held satisfactorily for several weeks, and construction work was fairly well under way, during which time the grouting of the remaining cells under

the unbalanced head proceeded with no definite results. Two or three incipient blows then occurred, one of which washed away the gravel berm against the inside of one cell. It was stopped by dropping sand bags and hay into the caving cell filling, and was definitely cured by drilling several extra grout holes. Another leak consisted of several mud boils, with a small flow of water, at the still ungrouted land end of the downstream arm and which, peculiarly, stopped completely of its own accord in about 30 min. About two weeks later, however, and before the grouting was completed near this point, the same boils reopened in earnest and within a short time the blows had increased until all equipment had to be removed from the cofferdam, and it was allowed to fill with water. Approximately 2.5 weeks of construction were lost while completing, under balanced head, the grouting of the land end of the cofferdam and for some distance into and upstream along the river bank where part of the blow occurred. When pumped out again, this corner of the cofferdam was perfectly tight and remained so. None of the minor leaks increased, in the upper arm or elsewhere, where grouting had been completed before unwatering. The general statement can be made, therefore, that at no point in either the second or third cofferdam were any but very minor leaks subsequently experienced where grouting had been completed under balanced-head conditions.

It was quite important to trace the flow of grout in certain holes that failed to build up pressure. Just prior to the blow under the downstream arm fluorescein dye was introduced into the holes. Quite soon it was detected emerging with clear water that had been flowing from open grout pipes at the bottom of the river end of the south steel pile cutoff wall, near its connection with the power house. The dye had traveled approximately 700 ft and presumably followed the same seams or "solutionized" bedding planes in the rock which a short time later caused the cofferdam to blow. The direction followed by the dye corresponded to one of the two jointing axes of the vicinity. These incidents illustrate the nature and condition of the upper rock strata, as well as the effectiveness of the grouting under these conditions, and they may suggest many desirable applications of this process.

Unwatering of the third cofferdam revealed several upper strata entirely reduced by solution to residual boulders in place and covering most of the power-house and tailrace location. Very few of the residual boulders have apparently moved from their original positions in the strata (see Fig. 28 submitted by Mr. Ross). The removal of the boulders revealed generally sound rock in the same stratum upon which the spillway had been built. No part of this stratum was undermined by open horizontal seams.

The remaining three spillway bays and the power-house intakes were founded upon this layer, as in the second cofferdam, and with similar preparation and cleaning of the rock. The excavation for the draft tubes of the generating units extended some distance below the previously described shale seam, beneath which the bedding was tight and was water bearing only to a negligible extent under the approximately 95-ft head existing at the bottom of several 36-in. drill holes.

Water testing proved all exploration holes to be remarkably tight, subsequently taking very little grout. The general nature of the power-house

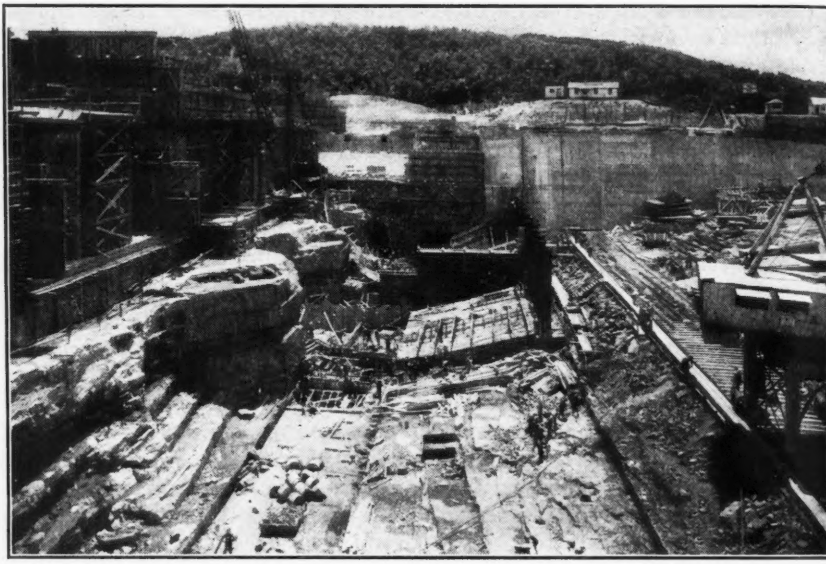


FIG. 40.—FACING SOUTH FROM THE SOUTH SPILLWAY TRAINING WALL, SHOWING POWER-HOUSE EXCAVATION

foundations is seen in Fig.40. The double shale is readily discernible. A complete pattern of holes was drilled and grouted under the turbine placements and entire intake to insure maximum bearing and to block off any small seams.

GROUT CURTAIN

A grout curtain consisting of high-pressure and low-pressure grout holes was extended across under the entire upstream face of the dam, including the intake, spillway, and upper miter sill of the lock. The low-pressure holes were drilled by wagon drill to a point below the shale, in two rows. One row of holes was drilled at approximately 15° inclination from vertical in one direction parallel to the axis and at 5-ft centers, whereas the second row was 2 ft downstream, spaced at 10-ft centers and equally inclined in the opposite direction, forming a pattern by which any vertical joint crack existing between the shale and the surface would be pierced in one or more places by grout holes. The open seams encountered were predominantly horizontal bedding planes. The low-pressure holes or grout curtain were grouted at pressures of approximately 25 lb per sq in. before concrete was placed. Displacement gages and a wye level were used at numerous points to detect any possible heaving of the rock strata. No displacement was detected at any point, although a very slight displacement probably occurred in one spillway bay where grout from high-pressure holes was found emerging from certain dowel holes and seams in the rock downstream from the spillway apron. Subsequently, a 36-in. drill hole immediately below the spillway baffle disclosed a layer of grout less than $\frac{1}{16}$ in. thick on top of one of the shale members.

The high-pressure holes were drilled through previously embedded pipes after the spillway weirs and floor of the intakes were placed, and were grouted at pressures of 50 to 60 lb per sq in. The quantity of grout taken by both low-pressure and high-pressure holes was very nominal except in isolated cases where there were very small open seams. Drain holes downstream from the grout curtain and under the spillway apron and draft tubes were drilled after all grouting was completed.

Steel Sheet Pile Cutoff Under South Earth Embankment

Investigation and correction of seams and caverns in the rock at the bottom of the steel pile cutoff under the south earth dike developed into a major operation. A temporary row of steel sheet piles was driven about 12 ft upstream from the cutoff piling, forming a sheathed trench as shown in Fig. 41.

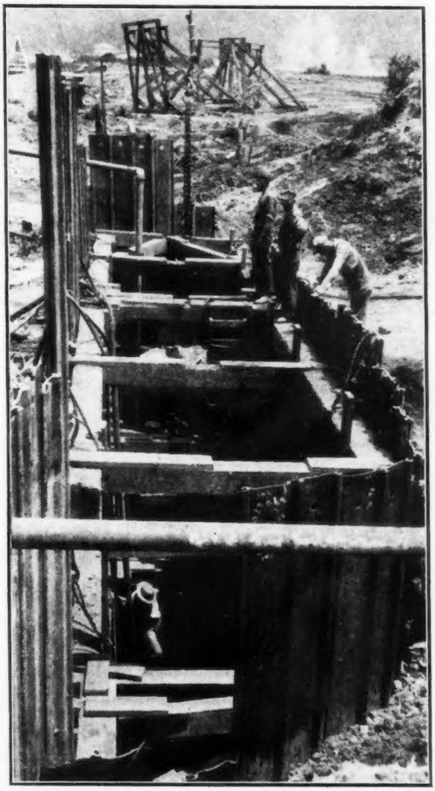


Fig. 41.—Excavating the Trench Along the Steel Pile Cutoff to Correct Seams and Solution Channels Beneath the Rock Floor

The first section of the trench was opened at a typical location where the piles had seated very irregularly. As soon as the pervious gravel strata were reached, water came in copiously, and when the excavation encountered the rock the remnants of one or two rock strata were disclosed which had been "solutionized" into nests of residual boulders with the spaces between filled mostly with very pervious water-bearing gravel (see Fig. 42). After removal of these boulders, the surface of the next stratum was found to be quite regular and relatively free from cracks, and the piles were reseated tightly on it after trimming off bent and injured ends. Jackhammer exploration through this stratum disclosed an open seam 4 in. to 6 in. deep, 3 or 4 ft beneath the surface. A trench was opened down to this seam, and jackhammer holes extending 10 to 12 ft deeper revealed no further seams. Furthermore, no seams were revealed by the deep shot-drill exploration holes, which had been increased in number until by that time, with the original prospecting holes, there were deep shot-drill holes approximately 100 ft apart. The trench in the rock was then filled with concrete to seal the seam, and a concrete toe wall was cast along the upstream side

of the piling about 3 ft high, except where required to be higher to cover split interlocks of the piling. The trench between the sheet piling was backfilled with puddled clay and silt for a number of feet, above which silt and fine sand were used. The temporary piling was finally pulled and reused. Small shot

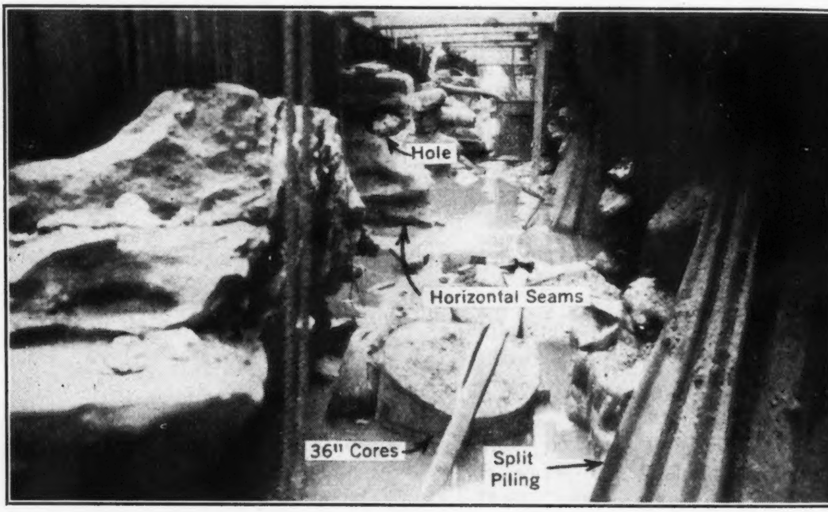


FIG. 42.—TYPICAL CONDITION AT THE BOTTOM OF A CUTOFF, SHOWING NESTS OF RESIDUAL BOULDERS

drills were used constantly along the axis of the cutoff in seeking out additional information.

As additional stretches of trench were opened, where the piling was most uneven, a variety of conditions was found. At many places where the piling tops were smooth and regular, and the bottoms were seated square and watertight upon smooth, apparently sound rock, the worst seams were often found beneath, at depths from 10 to 25 ft. This caused the trench to be opened for the entire length of the cutoff, and a technique was developed, which resulted in greater progress and economy.

In the various sections of the trench there were usually residual boulders to be removed, after which jackhammer, wagon-drill and deep shot-drill holes were sunk, sufficient (together with the preliminary holes) to determine the existence and depth of any open horizontal seams. Horizontal seams at shallow depths were corrected by close drilling and trenching, and later backfilling the trench with concrete. At times the top stratum would have longitudinal joint cracks which saved part of the drilling, and transverse joint cracks and "chimneys" into some open seam below. In one chimney concrete was found which had been placed by gravity through 6-in. pipes from the surface of the ground and at some distance away, horizontally. Such "chimneys" influenced the general ground-water behavior and invited sinkholes in flood plains. A number of the shallower seams opened into seams 2 or 3 ft deep which were only partly filled with sand and gravel. As pumping in sumps

continued, the general water table would soon drop until the water no longer came into the trench at the top of rock. Due to "chimneys" and vertical seams outside of the trench, it would run in freely through the lower horizontal seams, to the capacity of one or two 10-in. pumps. All manner of weird variations of structure and drainage were found.

Where it was not economical or feasible to open-cut the rock down to the seams, they were entered from the end in some instances, or through a series of 36-in. drill holes, and cleaned of all gravel, sand, and mud. Tight barricades consisting of bags of concrete were placed in the seams upstream and downstream, beneath and parallel to the steel pile cutoff, with 6-ft or 8-ft space between the barricades. The rock floor and roof were cleaned, and roughened if necessary, and a permanent concrete plug or wall was placed through the 36-in. drill holes. Check holes were afterward drilled through the roof and plug, and tested, to insure that the seam was tightly filled.

The number of deep 3.5-in. shot-drill holes was increased progressively until finally they were from 12 to 16 ft apart throughout the length of the entire cutoff. They reached well beneath the shale, and frequently much deeper.

The large seams were always found above the shale, although at two or three places the faulty condition extended to, and included, the shale. One such place, when cleaned out, developed into a cave approximately 10 ft high, under and roughly parallel to the cutoff, with about 20 ft of sound rock overhead. This cave had numerous branching arms and connecting seams. Figs. 43 and 44 are typical of the many horizontal seams encountered, which were usually of considerable extent, both longitudinally and laterally to the cutoff wall, having random branch caves and openings of unknown extent. In Fig. 44, light rays in one 36-in. hole indicate an adjoining hole. Five men in a line passed bags of concrete to build forms for a plug in Fig. 43. Fluorescein dye introduced into ground-water observation wells of perforated pipe beyond the upstream and downstream toes of the earth dam, and into several small sinkholes at the left abutment, appeared in the pump sumps within a few minutes by way of such typical seams as those illustrated.

For some distance along the cutoff there were two—and even three—seams, each 2 to 3 ft high, one below the other, the lowest one being about 25 ft below the top of rock. Workmen entered the seams by 36-in. drill holes (approximately 12 ft apart), to clean them out and place the barricades for confining the concrete plugs. An interesting game ensued in placing the concrete of the plugs and backing out with the pumps in such a manner that all concrete was placed "in the dry" and had opportunity to set without damage by flowing water.

Since a difference of head of 3 or 4 ft had developed above and below the cutoff due to diversion of the river over the spillway and the sealing of all other communicating seams conducting ground water, a considerable cross flow was expected at the point of final closure which was to be the pinching out end of the cave previously mentioned. The cave itself had been treated by open-cut methods from an adjacent open pit. To close the remaining 25-ft gap of low seam successive and interlocking holes were drilled 36 in. in diameter, allowing

the ground water to rise and practically come to rest before filling each hole full of tremie concrete, then pumping down again and drilling the next hole so that a segment of the concrete filling of the last hole was obtained in each succeeding core. This was a slow but necessary process as the concrete ap-

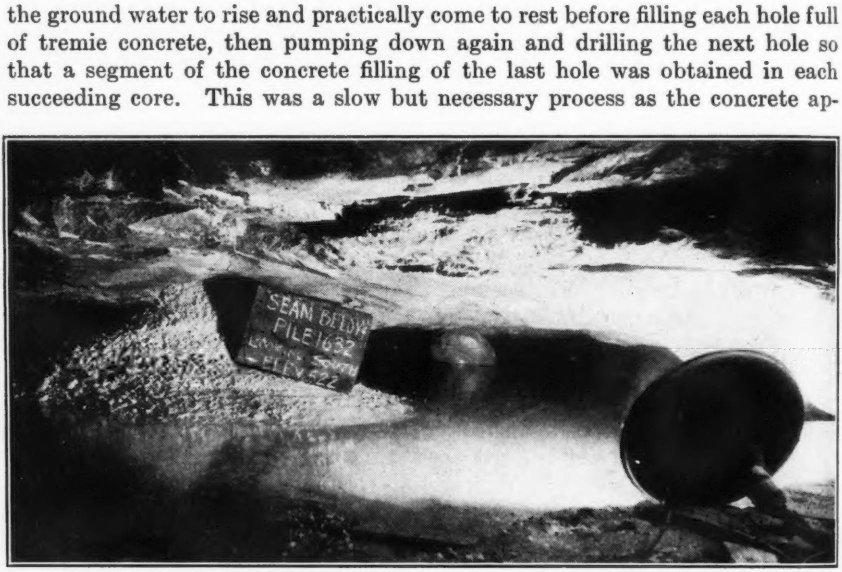


Fig. 43.—Horizontal Seam of Considerable Extent

Fig. 44.—Typical Seam Showing the Concrete Bags in Place for Forming the Concrete Plug Under the Cutoff

parently did not spread out in the seam very far from the bottom of each hole, or else it was washed away. An effective practice in this operation was to drill the 36-in. holes 3 or 4 ft below the seam to form a catch basin for sand and gravel, as well as a pump sump, thus assuring a full and solid concrete section

across the seam. Check holes were then drilled with a smaller core drill in the intersegments of the 36-in. holes to insure soundness and watertightness in the concrete filling. An expanding steel cylinder and other equipment and ideas had been devised, such as the possible use of 36-in., very thin, metal concrete buckets, or the use of a rock core lowered in heavy canvas sack later to be filled with grout, in case the method had not been successful.

As a final operation in all sections of the trenches, wagon-drill holes were sunk, 30 ft deep on 24-in. centers, along practically the entire line of the cutoff to avoid overlooking small local seams or defects. These holes, the 3.5-in. shot-drill holes on 12-ft to 18-ft centers, and jackhammer holes, were all fitted with pipes and grouted to refusal in order to seal the small horizontal seams, both above and below the shale stratum. At the pressures used (40 to 50 lb per sq in.) the neat-cement grout traveled considerable distances in very small seams throughout various parts of the work. Any horizontal seams small or large, at bedding planes, usually could be depended upon to show up in several adjacent holes, and it was nearly impossible for them to escape correction. Certain "solutionized" vertical jointing seams, not definitely sealed otherwise, were cut off by concrete-filled 36-in. drill holes. The predominant open seams, however, were horizontal, and open vertical seams were seldom encountered except where they were visible in the topmost stratum.

Guided by the special experience and detailed knowledge of conditions gained in successive trenches, as a precautionary measure the first trench completed was subsequently drilled on 5-ft centers to a depth well below the shale seams, and regrouted.

Except for final grouting and backfilling, the last operation in all trenches was the construction of the concrete sealing wall along the bottom of the piling, filling all trenches and large drill holes, and sealing all splits in the piling interlocks by carrying the wall upward as necessary.

A flashlight beam could be thrown for considerable distance in some of the seams and the system of seams was proved (by fluorescein) to extend entirely through under the earth-dam section. Therefore after filling the reservoir there was a possibility that sinkholes might be formed in the flood plain or in the toe or slope of the earth dam by flood-plain material being carried down through "chimneys" into voids in the seams although all seams were completely blocked at the cutoff as herein described. To obviate this possibility, a system of holes was drilled both upstream and downstream, into, and in the vicinity of, the known open seams, and all voids that were found were filled by grouting with a mixture of fine flood-plain sand mixed with cement in proportions of approximately 1 to 4. This mixture was found to travel well and to be capable of setting quite firmly under water. During the time that some of the trenches were open, considerable ground was lost into the trench, at places, due to heavy rains and also to movement of ground water. These losses occurred under the sheet piling at boulders and split interlocks, and into seams and cavities outside of the trench, resulting in some open holes to the surface. To guard against voids, a water jet was used to feel out all voids which did not run to the surface, and fine sandy silt was added and jetted into place to refusal. The jetting was done carefully as the puddled backfilling was placed in the trench in such

manner as not to displace the piling in either direction, although very slight displacement did occur at two or three points, in an upstream direction.

As the work progressed and plottings of the original exploratory borings were studied with respect to the unexpected conditions found (which required the extensive treatment described herein), it was seen that preliminary holes 100 to 200 ft apart were entirely inadequate in soluble limestone as a basis for proper deductions and design. The horizontal extent of the seams is not made apparent; and only an occasional hole, if any, will give any previous hint of the worst conditions to be encountered. The logs of these holes can be diagnosed properly only with the help of additional adjacent holes. In regions of soluble limestone, the preliminary holes, for proper design and estimating (although seemingly excessive cost and time are involved) should be not more than 25 ft apart along the axis expected to be adopted and many of the holes should be very deep ones. All holes showing open or mud-filled seams, appreciable loss of core, or loss of much water under test or during drilling, should be supplemented by adjacent holes at 5-ft to 10-ft spacing for the purpose of delimiting the extent of such seams in all directions.

The equipment usually available or supplied for sinking test pits, and for pump testing therein, is often very inadequate for the purpose of obtaining full and essential information. It is believed that, where preliminary borings show defects in the rock, one of two procedures is indicated: either (1) fully adequate equipment and methods should be used for sinking test pits, safely and satisfactorily; or (2), casings, at least 36 in. in diameter, should be driven to rock, with large holes drilled in them and with pumps of the proper type and sufficient capacity to permit examination and to determine the full water-producing capacity of such seams. The writer believes that fluorescein tests should also be made, from well points or other test pits some distance away. A variation that is sometimes effective is to fill certain pits or pipes with water and observe the amount and time interval of effect on water levels in adjacent pits and wells. Added information may often be had by use of fluorescein, sal-ammoniac, or other chemicals. Most satisfactory, expeditious, and economical results can be obtained under adverse conditions by a 36-in. drill, or larger, sinking a casing through the flood plain, and drilling into the rock well below all pertinent seams to form an ample catch basin for sand and gravel during pumping tests and to permit entrance and inspection of the lowest seams. The casing seen standing in Fig. 28 (presented by Mr. Ross) was such a hole drilled through the flood plain on the center line of one of the generating units; and another such hole was drilled through a crib set in the river. The pumping will frequently require a 10-in. well-type pump, or larger, and it will usually promote speedy results and economy in the end to employ equipment capable of coping fully with conditions.

With full information available in advance as to the exact conditions which would be encountered in perfecting a cutoff at this site, there would still doubtless have been differences of opinion as to whether the work should be prosecuted by open-cut or by sheathed trench methods. It is the opinion of the management on this job, considering the particular conditions existing, that the method used was somewhat more economical, and quicker, and fully as certain of positive results as if an open-cut had been made to rock.

Foundation of Rolled-Fill Sections

Design specifications required that the top of the steel pile cutoff should extend a minimum of 5 ft into the rolled-fill embankments. Comparative estimates indicated the economy of selecting the pile lengths so that they would drive approximately flush with the original ground after the stripping had been done. A cutoff trench 5 ft deep and approximately 12 ft wide on each side of the piling was excavated and the rolled fills were started by compacting the backfilling material of these trenches with sheepsfoot rollers, under laboratory control. In this way it was possible to reach soil of maximum natural compaction, and was considered preferable to shorter piling and a deeper and narrower trench with puddled backfilling. The most homogeneous results were obtained by making these trenches of liberal width.

It was found necessary to puddle the deep, narrow sheathed trench which had subsequently been excavated to the bottom of the cutoff piling for correction of boulders, cavities, and seams. However, the material used for puddle (except near the bottom of the piling) permitted fairly free draining and had considerable time to settle and shrink before the rolled fill was started.

Three years of observations on the rolled fills indicate that settlement had substantially ceased early in 1940, with a maximum of approximately 5 in. and 4 in. in the approximate 70 ft from the top of the rock floor to the crest of the dam, for the north and south fills respectively, there being about one third of the settlement in the 35 ft of rolled fill, and approximately two thirds in the 35 ft of underlying flood-plain materials. The results were approximately the same for both the south embankment, with the 12-ft wide puddle trench to the rock floor just upstream from the cutoff piling, and for the north side where such a trench was not necessary.

The rolled materials varied from the finest of river silt and sandy silt to residual clay, the latter being used for the plug next to the lock wall. Here the total settlement is approximately $1\frac{3}{4}$ in. for a wedge of rolled residual clay 70 ft high resting on the rock on its lower edge, which is about 20 ft thick, the other supporting materials being the lock excavation slope of fine river silt and the end slope of a section of rolled embankment of fine silt. However, on the south side of the river, for a switchyard electrical cable tunnel resting on a similar plug of sandier river silt adjoining, but not a part of the dam, there has been a maximum total settlement of approximately 4 in.

Efficacy of Cutoff Provisions and Corrective Measures Applied to Seams and Cavities, as Indicated by Ground-Water Observations and Other Evidence

Since the pool was raised to reach normal operating levels in the latter part of January, 1939, the behavior of the ground water downstream of the north and south embankments has been studied by means of a number of observation wells. Time graphs have been made of the periodic observations, and ground-water contour maps have been prepared at intervals of 5 to 7 days.

On the north side of the river, a general seam of open gravel 3 or 4 ft thick was exposed on top of the rock in the lock excavation and from evidence of

pits and drill holes the gravel tapers out irregularly near the north abutment. The wells terminating in the gravel respond to tailwater variations in proportion to their distance from the river; a few, in more impervious materials, respond to rainfall; and none respond noticeably to ordinary pool-level variations. One well pipe near an original spring had been flowing for about two years. When the pool was filled it was decided to put a riser on this pipe, and the water level in it rose about 10 ft above the ground surface. The head on this well had not been determined previously. This effect (after much study and the addition of several wells, chemical analyses of water, temperature observations, etc.) has been proved to be purely artesian and sensitive only to rainfall, which probably percolates down the side of the mountain and produces pressure in seams in the rock, with which the one particular well came in contact. All observation well casings were perforated for 5 ft at the bottom and driven to rock.

Slow saturation of the north abutment apparently tilted and somewhat changed the original direction of ground-water flow. Possibly this change augmented the flow slightly, as the original spring mentioned, plus flow from the artesian pipe, increased by increments following certain rains, from a total of 5 gal per min to 25 gal per min over a period of 3 months and has held practically steady, at 24 gal per min, ever since.

On the south flood plain, at the beginning of the work in January, 1936, recordings of the water levels in all drill casings of the preliminary borings were made every day or two, and superimposed time graphs and river levels were plotted. Two major flood crests occurred in January, 1936, and one in February. Several of the wells very near the river followed the river fluctuations closely. The others rose very slowly with the repeated floods and were grouped within about 2 ft of El. 558 by the end of February, with little change through March, April, May, and June.

The original natural condition on the south side of the river in June, 1936, before anything has been done to change it, is shown in Fig. 45. The effect of pumping in the cutoff trench in September, 1937, with the cutoff partly completed and with the river partly constricted by the second-stage cofferdam, is also shown in Fig. 45. Fig. 46 demonstrates the remarkably slow response of the ground-water table to the filling of the pool in January, 1939 (if, indeed, the apparent response was not due to heavy rains, as suggested by well No. 25).

In other words, throughout June, 1936 (see Figs. 45 and 46), all wells were steady at approximately 11 ft above the river, which had been steady at El. 547, the extreme low water stage being 544.5. This high and flat water table was in contrast to the north flood plain and indicated little drainage downstream or toward the river.

In August, 1936, pumping in the first trench along the sheet pile cutoff began, and most of the wells were immediately affected, some drawing down as low as El. 520, or 27 ft below the river, which still held practically steady at El. 547. No direct inflow or very rapid infiltration from the river was indicated.

With certain trenches being backfilled and others pumped for many months, the behavior of the wells can only be interpreted by detailed study of all data. In general, until the first winter rise, the river held steady between El. 547 and

El. 548. A number of additional job-made, perforated, well points were driven to rock upstream and downstream from the dam. Some of these went dry during the pumping, indicating an area of free water table somewhere down in the seams and caverns leading to the pumps. At times some wells immediately upstream and downstream from the completed cutoff registered approximately El. 522, which elevation is seen on the placard in the seam shown in Fig. 43. The deepest pumping recorded was in the bottom of the largest cave where the water was drawn down to El. 507.5 at one time, which can be judged from the elevation "514" painted on the rock in Fig. 47. An interesting incident occurred during the final closure of this cave and seam. One of the 36-in. drill cores eventually brought to the surface the piece of stone on which the elevation had been painted. The two adjacent views are of the same cave, with a row of timber posts supporting the roof between. The three large dark voids are the openings of branch caverns, and the black slot near the bottom marks the location of a former shale seam. The arrows indicate the direction of flow of the entering water. El. 507.5 corresponds with the bottom of the general shale seam from which the shale here was missing for some distance, and which seam, being only partly filled with sand and gravel, was bringing water copiously from several directions. This was in November, 1936.

Two wells against the south abutment cliff rose promptly and decidedly at each rain and, after the filling of the pool, well No. 25 was found to register as low as, or lower than, any of the wells on the flood plain during dry periods. Two other wells nearest the river bank consistently held at or about the original El. 548 (see ground-water contour maps, Fig. 45). Occasionally, these wells, and all others, were tested for activity by bailing out or pouring a little water into them. They were situated in an area where comparatively tight clay and silt were known to exist along the south bank of the river.

The behavior of these two holes, the original test pits near the river bank, earth auger holes, the fact that all water flowing into the trenches was clear and cold, and the shape and extent of the pumping drawdown funnels in the water table, appeared to indicate that the flood plain was largely cut off from the river and acted as a large filter bed drawing its ground-water supply chiefly from precipitation, from upstream, and from the landward. Later data appear to support this conclusion. As stated, it was known that "chimneys" existed in the rock, and sinkholes existed near the south bluff, so that at times of river overflow the flood plain and seams would be filled quickly. Although two sinkholes later appeared in the borrow pit about 700 ft below the dam, present evidence shows that ground water and such leakage as exists do not drain readily either in a riverward or downstream direction. It may even drain partly southward and into the rock bluff itself, to some lower and unknown horizon (see shape of contours on map of March 26, 1939, shown in Fig. 45).

The ground-water time graphs for the pumping in one of the trenches in September, 1937 (Fig. 45), is one of the few instances suitable for illustration, since the remainder of the graphs are complicated by pumping in adjacent trenches. The accompanying ground-water contour map indicates the extensive drawdown funnel due to pumping, which extends to the south bluff, but is bounded riverward by the known region of tight ground near the river. At

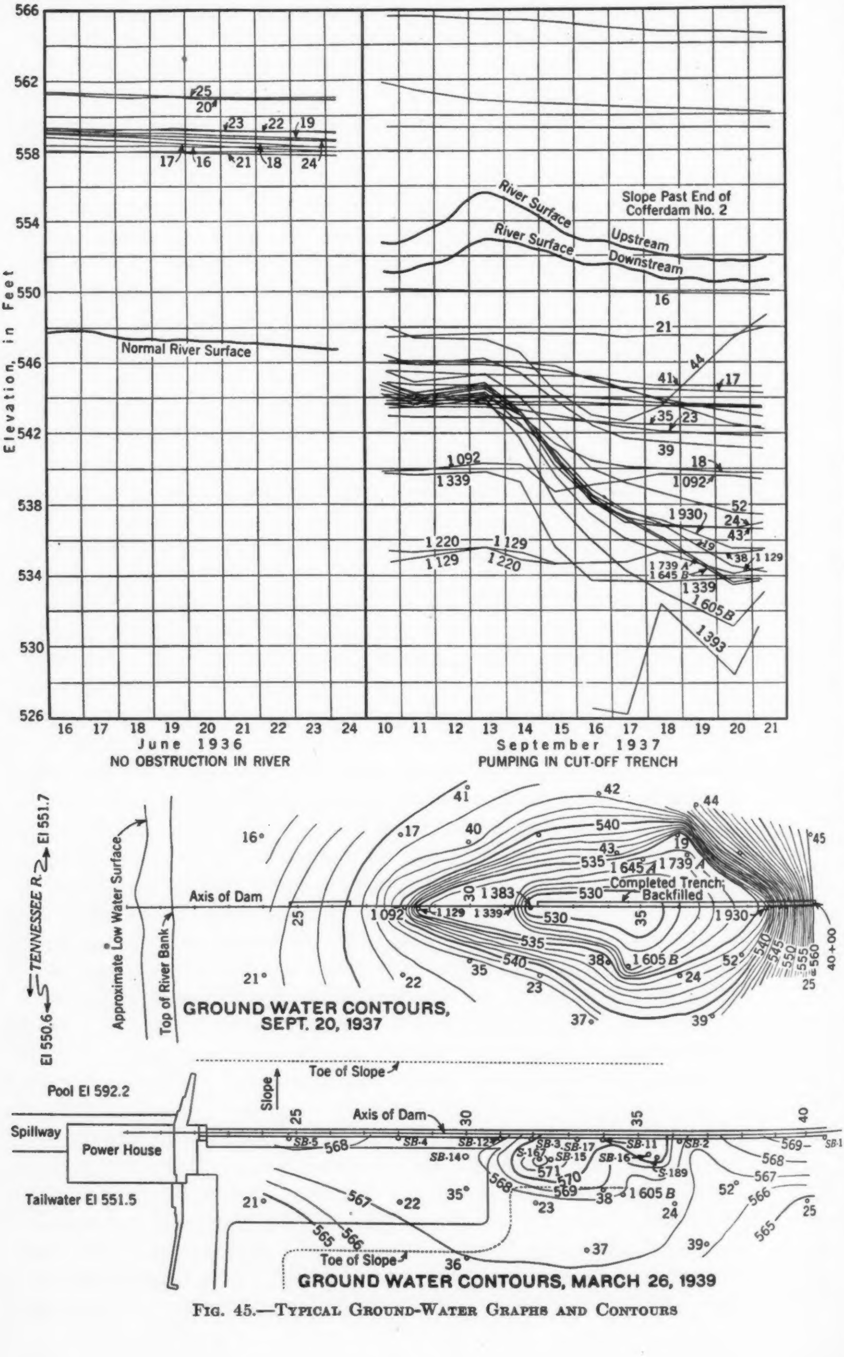


FIG. 45.—TYPICAL GROUND-WATER GRAPHS AND CONTOURS

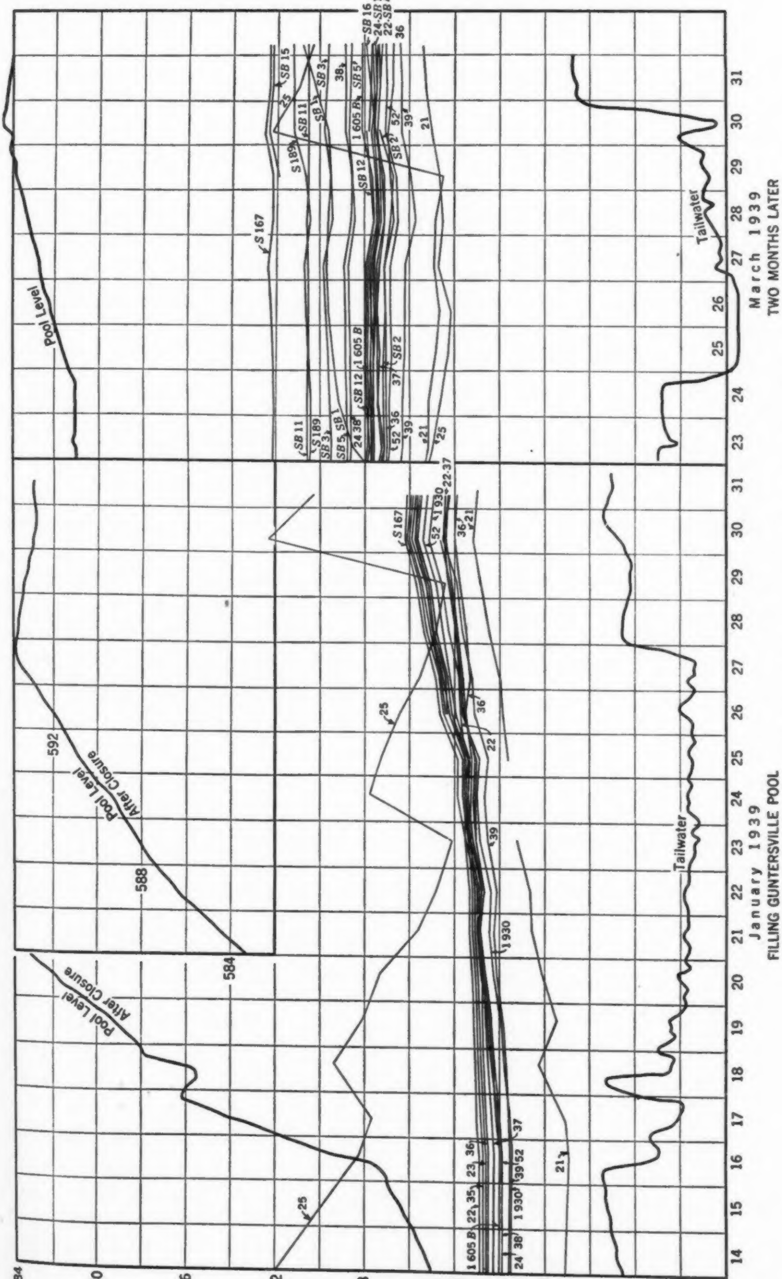


FIG. 46.—TYPICAL GROUND-WATER GRAPHS; JANUARY AND MARCH, 1939

this time approximately 1-ft drop in river level is caused by cofferdam No. 2. All wells are drawn down below river level except the "rain-water" wells at the south bluff (well No. 45 being defective).

Between December 14 and December 18, 1937, the upstream leg of the third and final cofferdam was closed, diverting the river through the spillway openings over permanent concrete weirs with tops at El. 555. This caused an initial drop of approximately 10 ft between pool level and tailwater. Simultaneously pumping in one of the trenches ceased and backfilling proceeded. All wells were grouped closely and sought normal elevations, rising at the rate of approximately 2 ft per month and ending the month at about El. 549.

The general ground-water rise through January and February, 1938, was approximately 2 ft per month. The pipes nearest the unwatered cofferdam No. 3, however, responded to the influence of that cofferdam until the blow on March 12, 1938, when water levels rose promptly and as promptly subsided when unwatering started again on March 29.

Pumping in the cofferdam affected only the nearest holes, all the others pursuing their gradual rise, with a spread of about 5 ft, between El. 554 to El. 559 at the end of March, and 557 to 563 at the end of April. This was still generally below tailwater indicating very slow infiltration which was apparently accelerated somewhat by March and April floods. The last open trench nearest the power house was backfilled during April, 1938; and at the end of May, the indicative wells ranged from 558 to 562, being between tailwater and headwater, and registered no rise during the month. At the end of June they were from 559 to 563, exclusive of the "rain-water" wells No. 20 and No. 25, they were still between headwater and tailwater; at the end of July, the elevation was 560 to 563, except as slightly affected by a small July flood; at the end of August, it was 560 to 564, having dropped slightly following the river; at the end of September, 558.5 to 562, still being slightly above headwater which was then at 560, and slowly falling with tailwater, which was at 554; and, at the end of October, 559 to 561, with headwater at 560 and tailwater at 554. During November, the head gates were set in place and the upper portion of the power-house cofferdam was flooded. All wells were between 561 and 563 at the end of November, with the headwater after the flood approximately at El. 562 and tailwater at 553. The wells were brought up by a suction dredge, pumping gravel from the cofferdam cells into a borrow pit which approached within 400 ft of the dam, thus affecting practically all wells. At the end of December, with the dredge still pumping into the borrow pit and with the water in the pit at El. 568, the wells were between 561 and 563 with headwater at 563 and tailwater at 555. The influence of the dredge pumping apparently was considerable in holding up the water table, but from the foregoing, it is apparent that the wells had held practically stationary for two months.

The foregoing summarizes ground-water behavior for the three years from the beginning of construction to the filling of the pool, which started actually on January 16, 1939. From Fig. 46, it is seen that the headwater rose from 567 on January 16 to 594 on January 27 and receded to 593 on January 31. (The variation from a smooth storage curve on January 17 and 18 was due to the opening of the spillway gates—reflected in the tailwater curve—to hold the

water until certain obstructions in the lower reservoir had been removed.) The wells rose from a close group average of 562 to 565 during this period, accelerated considerably by frequent rains and particularly by a precipitation of 1.42 in. on January 29. (The occurrence and relative quantity of precipitation can be seen by behavior of well No. 25.) In early February the pool remained approximately at 594, but rising tailwater and rains (such as 2.83 in.

FIG. 47.—CAVES IN WHICH DEEPEST PUMPING WAS RECORDED

on February 2, 1.37 in. on February 6, 1.89 in. on February 10, 0.99 in. on February 14, and 0.75 in. on February 19) brought the wells up in their risers to from 573 to 577 on February 19, with tailwater overflowing the flood plain at 574. Then the tailwater subsided rapidly, and the wells subsided about one half as fast to a close average of 568 on March 1. Although the water table fell approximately 7 ft from February 19 to March 1, that it was free from headwater influence is not fully conclusive since headwater also fell 3 ft during the same period. Further rains in early March, however, brought both tailwater and water table up approximately 2 ft by March 11, after which, with the pool held steady by gate manipulation, except for rising and falling 2 ft over a three-day period, the wells all dropped an average of 3.5 ft to approximately El. 568, with a tailwater drop of 5 ft. The wells held steady until (see Fig. 46) on March 24, the pool was ordered raised from 591 to 594, which it reached on March 30. Although there were rains totaling 2.03 in. during this period, the ground-water table held steady at an average elevation of 568.

Until this time (March 31, 1939) all available data indicated strongly that the observation wells were not influenced by ordinary pool fluctuations. However, the general water table had risen slowly from the average El. 562 to El. 568 (approximately) subsequent to filling the pool, the final lift of the pool

being 30 ft, from El. 564 to El. 594. No springs or seepage had shown on the surface near the dam; nor did any appear until, in August, 1939, at several points, approximately 800 or 900 ft downstream in the bottom of a borrow pit, slightly above the level of Wheeler Pool, a seepage flow of approximately 50 gal per min appeared. Chemical analysis of this seepage indicated that it is probably not from the reservoir, or at least to any great extent. Except for this instance, no evidence of springs or seepage came to the surface.

There was considerable evidence to support the view that the ground water moved downstream very slowly; and, since there were no springs or surface seepage except for the 50 gal per min in the borrow pit, seepage past the cutoff must be nominal. However, without direct determinations of direction or rate of the movement of the ground water, there could be no more than considered opinion as to the quantity of seepage past the cutoff wall.

Ordinarily, the absence of surface seepage, or springs at some point not far below the dam, would be sufficiently reassuring. However, there was one phenomenon suggesting further study: After the pool had been filled, and after several subsequent rains, one early observation well (No. 167) began to rise slowly above the others, after about 1.5 months, until by the last of March, in interpolating ground-water contours from the rather widely spaced wells, it caused a "bump" about 4 ft high above the general water table under the toe of the dam. This was corroborated partly by certain adjacent wells. Since practically no water was being picked up by the longitudinal toe drain of the dam, this was interpreted as an artesian effect somewhat similar to the flowing pipe downstream from the north dike, indicating pressure in rock seams, possibly from, or augmented by, deep underseepage from the pool. The height of this "bump" remained practically stationary at El. 573 for some time, but later rose again slowly. Meanwhile, additional wells had been driven as a check and to aid in interpolating contours, as several original wells had been lost during construction and by silt-cement grouting. The new pipes corroborated the "bump" and defined it more clearly, until well SB–22 was driven in the lower slope of the fill and brought in a flow of approximately 5 gal per min at the casing top which was at approximately El. 574. An extension on this casing registered a static elevation of 577.7 on October 3, 1939, raising the top of the "bump" considerably and, with the other wells, moving it further downstream.

At the time, it was thought that well SB–22 had been driven very close to a "chimney" in the rock, from which seepage water had been rising and diffusing in all directions. Flowing water had been encountered in sand and gravel at about 19 ft below the bottom of the rolled fill as the pipe was driven. It continued after the pipe was driven to rock, through the perforated lower 5 ft, running clear, with no sand, after driving ceased.

A rough attempt was made (with no positive or interpretable result) to determine the direction and rate of flow by means of electrodes in several available wells and a charge of sal-ammoniac in well SB–22. A fan-shaped set of wells, Nos. SB–27, SB–28, SB–29, and SB–30, were then driven to rock at approximately El. 533.7 downstream of the toe of the dam opposite the "bump," for measurements by means of the Slichter apparatus[4] previously constructed

[4] "Field Measurements of the Rate of Movement of Underground Waters," by Charles S. Slichter, *Water Supply and Irrigation Paper No. 140*, U. S. Geological Survey, 1905.

and used in 1935 at Pickwick Landing Dam. The Slichter apparatus, at once, gave a rate of flow of 7 ft per hr in a downstream direction. The pervious material here was a layer of gravel 3 in. to 4 in. thick lying on top of rock. From approximate calculations, this would indicate a seepage of approximately 8.75 gal per min per 100 ft of periphery of "bump," which for the apparent length of active periphery would result in the rather nominal total seepage of approximately 53 gal per min.

Slichter tests, checked with an electrical "megger," at a point near the south end of the fill, showed only a slight drop in resistance but nothing interpretable. At one installation on the north flood plain, near the toe of the fill, approximately 100 ft north of the lock, nominal movement was indicated. All installations had been selected, from the ground-water contours, as being most likely to register positive water movement.

It could then be concluded reasonably that probably the only place where there was movement of importance was in the vicinity of the "bump" in the south flood plain. This seemed logical since the "bump" is immediately downstream from the large cave which was found and blocked with a concrete wall under the steel-pile cutoff wall and the general area where the most difficulty was experienced in correcting various seams and caverns. There were arms of these caverns pointing in a general downstream direction.

It had been intended next to drill some of the Slichter holes down into the rock, expecting to strike some of the seams and to determine whether there was measurable movement of water therein. It was not known whether this had ever before been attempted or accomplished. When drilling began in rock, hole No. 28 (approximately 75 ft west of well SB–22) struck a 1-in. seam 12 ft below the top of a rock which produced a flow of 20 gal per min at the casing head, bubbling up 2 or 3 in., with a little sand at first. After a rain, and possible self-cleaning, the flow increased and remained steady at 30 gal per min. Almost instantly after this flow started, the water level in SB–22 and several other pipes forming the "hump" began to fall at a rapid rate (0.8 ft in the first 3 hr). Quite soon the rate decreased and in about 48 hr had become stationary, with a maximum permanent lowering of the previous "hump" of approximately 1.6 ft. It was realized that by use of "packers," water from rock seams could be controlled if desired; and that the "hump" or pressure zone under the toe of the earth fill could probably (if desired) be reduced further. Therefore, holes SB–27, SB–29, and SB–30 were drilled into the rock. No seams or water was encountered at comparable levels in these holes, and it was decided to try between hole SB–28 and SB–22 near the foot of the dam. All subsequent pipes were not perforated and were seated in the rock before core drilling began. The purpose of thus seating the pipe was to prevent unknown quantities of water, if developed, from escaping without control into pervious gravel, it being desired not only to measure but to control all water liberated from the underlying seams.

Hole SB–46, approximately 46 ft west or downstream from SB–22, promptly brought up 25 gal per min from 12-in. and 24-in. seams, 2.6 ft and 6.9 ft, respectively, below the top of the rock at El. 534.3. Thus hole SB–22 and the bump were lowered, quickly and permanently, 0.8 ft more. Hole SB–48, only 12 ft

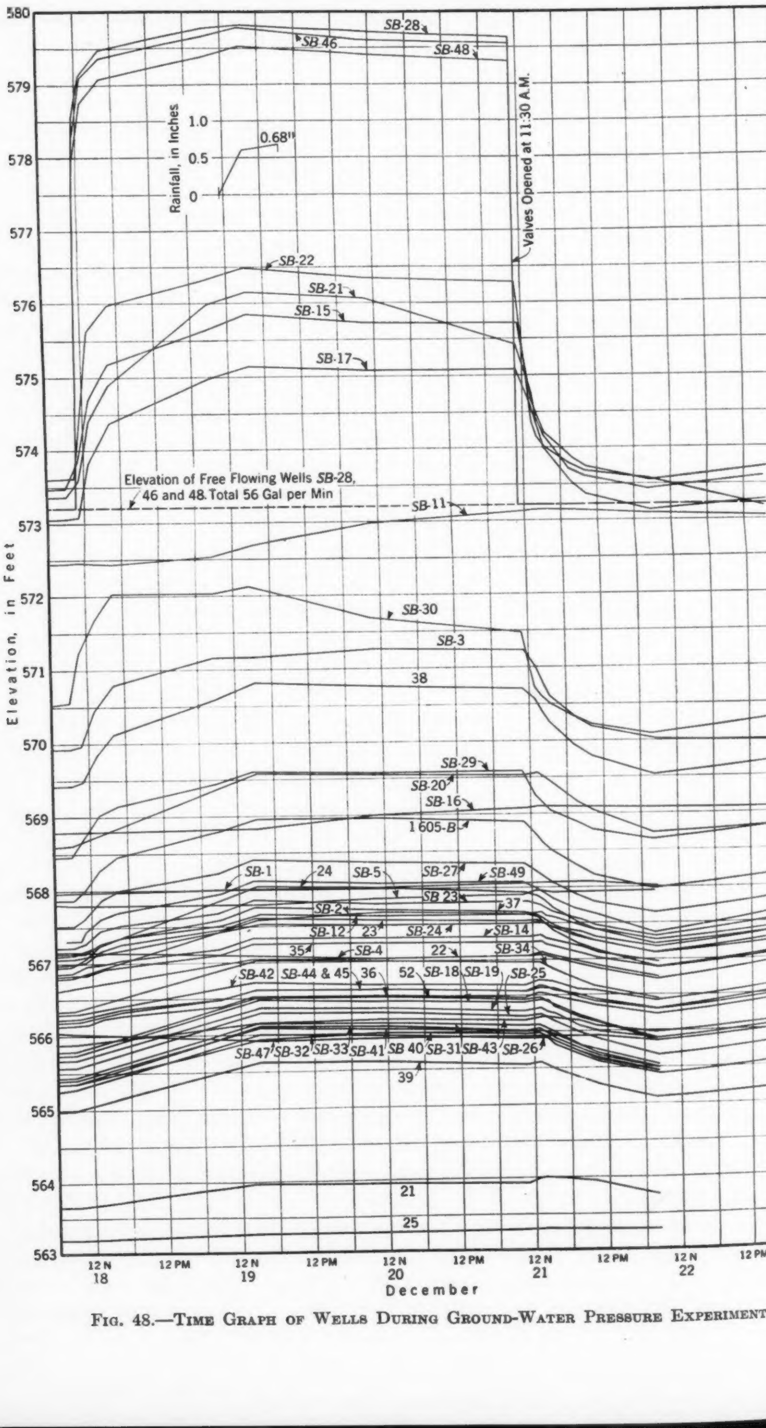


Fig. 48.—Time Graph of Wells During Ground-Water Pressure Experiment

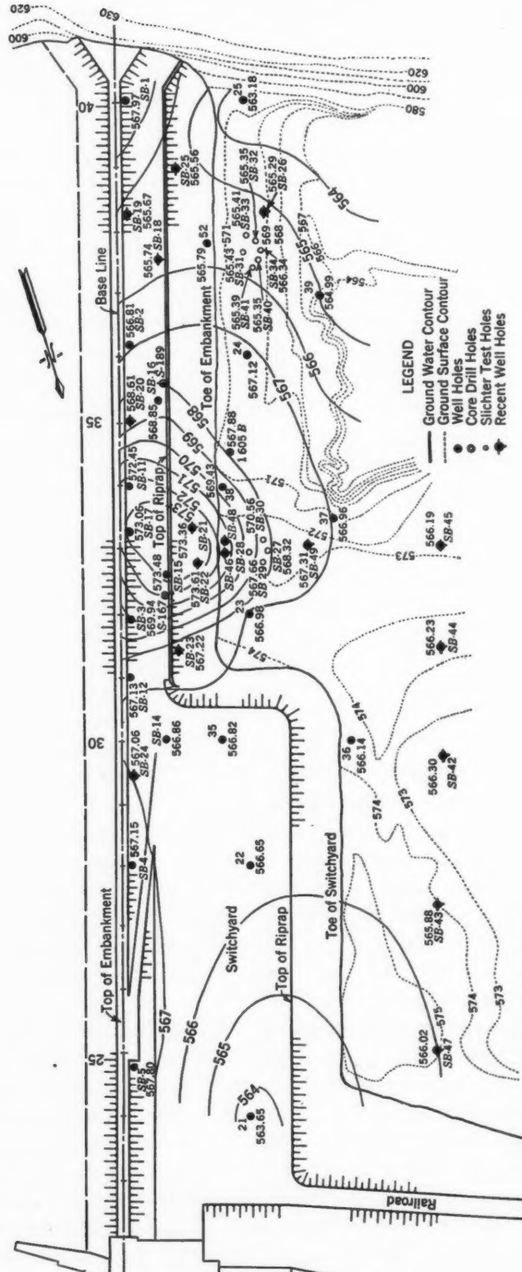
south of SB–46, struck a flow of about 30 gal per min in gravel at the same level as the water-bearing seams in hole SB–46. This new flow also robbed most of the flow from SB–46, until SB–48 was driven to, and seated in, the rock. Drilling SB–48 into the rock produced only 4.5 gal per min from a 2-in. seam at approximately 1 ft below top of rock at El. 521.3.

By now the complete interconnection of all flowing pipes had been established as well as the fact shown by holes SB–46 and SB–48 that the seams in the rock, and gravel above the rock, were interconnected. Drilling in hole SB–46 had muddied the water from hole SB–28 and other like phenomena.

By now there was a total flow, from all pipes, of approximately 56 gal per min (SB–28, 30 gal per min; SB–46, 21.5 gal per min; and SB–48, 4.5 gal per min). It was also evident that, whether the water came from the reservoir or other sources, the total of 56 gal per min was a considerable proportion of the supply to the area, sufficient to affect a great number of wells and lower the "bump" on the water contours rapidly and to a considerable extent. It seemed reasonable that this flow might be practically all of the underseepage (possibly augmented somewhat from other unknown sources) and that the "bump" might represent the head necessary to overcome friction in forcing the water to and out of the wells. Conceivably, until relief was provided by the first flowing well, SB–28, the "bump" had been rising very slowly centered near SB–28 as the apex, due to accumulation of water (since escape channels appeared restricted), in an attempt to force the seepage through other escape channels.

The foregoing suggested an interesting experiment. Tees with outlet gate valves and risers were added to the flowing wells. All of the valves were closed at once and readings at close intervals were taken on all pipes constituting the "bump" and most of the entire system of pipes in the south flood plain, with the result shown on the time graph in Fig. 48. Fig. 49 consists of two groundwater contour maps interpolated from the existing wells just before and shortly after the valves in the flowing wells were closed. It will be noted (see Fig. 48) that the level in the pipes most affected rose nearly to full height in approximately one hour and that practically all pipes, wherever situated, were at least slightly affected. After most of the pipes had come substantially to rest, the valves were re-opened and the "drop down" of the principal pipes was even more remarkable, being largely accomplished in 15 to 30 min and resulting in a concave graph. A water stage recorder had been installed on SB–22 so that the exact shape of this curve was obtained.

Roughly, the foregoing—and particularly the shape of the drawdown curve—would seem to indicate that, (1) the wells that were active and sensitive during the experiment connect (*a*) with a rather small and fairly tight reservoir, consisting of interconnecting seams and gravel, or (*b*) a larger and much tighter reservoir; and that the supply to this reservoir (presumably underseepage) cannot be greatly in excess of the 56 gal per min which was checked and then released (by closing and opening the valves) to produce the recorded effects; or, (2) that whatever supply exists is escaping through a tight and rather small conduit or system of channels of small capacity. Indication (2) seems less tenable than indication (1) since pipes over a wide area are evidently in connection with this channel. In any event the amount of flow through these



(a) BEFORE CLOSING VALVES

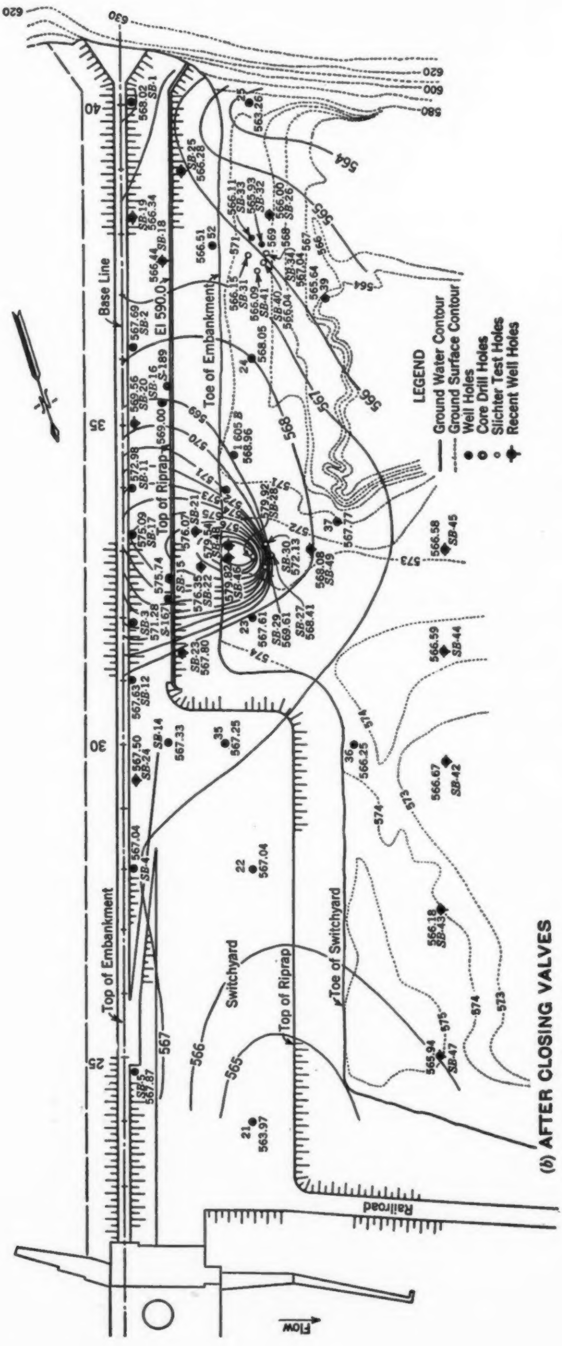


FIG. 49.—WELL-HOLE WATER ELEVATION READINGS BEFORE AND AFTER CLOSING OF VALVES IN FLOWING WELLS SB-28, SB-46, AND SB-48

pipes cannot be many times more than the 56 gal per min which were manipulated. This channel (to explain the connection of so many wells) might consist of a very extensive system of small seams in the rock.

Further evidence tending to prove the tightness of the underground reservoir or system of seams in the rock and lack of outlet downstream is afforded by the facts (1) that the ground-water level originally stood for considerable periods as much as 10 ft or 12 ft above the river level and (2) that during the latter part of the clay cement grouting, fluorescein was forced out of certain wells in this area, which had lain underground for approximately two years.

Numerous open vertical-joint cracks and vertical "chimneys" have been found wherever bedrock has been exposed having seams underneath. It had previously been assumed that perforated pipes driven into gravel lying on the rock or with drill holes into seams registered practically identical pressures. This was supported by several actual cases of adjacent pipes. However, it is evident that this is not true in the vicinity of the "bump." It would seem, therefore, that this system of open seams and caverns (with "chimneys," etc.) is rather definitely limited in a downstream direction and points to the existence of a comparatively tight "reservoir" rather than a channel. Chemical analyses of the flowing wells to date do not determine their source, definitely, as reservoir seepage.

Various conclusions may be drawn from the circumstances described, but it is the writer's opinion, based upon a long study and day by day familiarity with all evidence and conditions (and particularly since the south flood plain has the ability to hold the water table comparatively level and high above tailwater for long periods) that there is very nominal seepage past the cutoff and that the plugging and connecting of caves and seams, etc., along the line of the cutoff is proved to be exceptionally complete and effective.

FOUNDATION EXPLORATION AND GEOLOGIC STUDIES AT CHICKAMAUGA DAM

By Portland P. Fox,[5] Jun. Am. Soc. C. E.

Synopsis

Engineers and geologists seldom encounter more difficult foundation problems in the construction of a large dam than those which prevailed at Chickamauga Dam. The rock forming the foundation of the dam consists mainly of thin-bedded, pure to argillaceous limestone with thin interstratified beds of bentonite and shale. The rocks are intricately folded and faulted, and are quite cavernous, especially in the zones of structural weakness. All of the foundation problems, including the heavy overburden, are related, directly or indirectly, to the structure and solubility of the rock. By carefully logging, correlating, and plotting all data made available from many hundreds of core-drill holes of both small and large diameters, it was possible to reconstruct the complex foundation conditions, in detail, in advance of actual excavation. This detailed geologic work aided the engineers in determining not only the depth to which excavation was necessary, but also the extent and character of foundation treatment that were necessary.

Introduction

Chickamauga Dam is on the Tennessee River, 471 miles above its mouth and 7 miles upstream from Chattanooga, in Hamilton County, Tennessee. It is the first dam to be constructed on the Tennessee River in the extensive physiographic province known as the Appalachian Valley. This valley lies between the Appalachian Mountains on the east and the Cumberland Plateau on the west and has an average width of 50 miles from central Alabama to New York. It is sometimes spoken of as the "Valley and Ridge Province," or the "Great Valley of East Tennessee." Geologically, it is characterized by a great variety of deformed strata of Paleozoic age. Rocks differing in their resistance to erosion have given rise to reefs, shoals, and pools in the Tennessee River as it meanders southwestwardly across the strike of the various formations. In the vicinity of Chickamauga Dam the river follows a meandering course down a flat-bottomed valley of 0.5 to 1.5 miles in width.

Previous to the creation of the Tennessee Valley Authority (TVA), the U. S. Army Engineers had investigated two possible dam sites in the immediate vicinity of Chickamauga Dam site by diamond-core drilling methods, one just below the mouth of South Chickamauga Creek (mile 468.7) and another 0.9 mile above the present Chickamauga Dam (mile 471.9). The TVA considered the lower site (mile 468.7) to be not feasible for a high dam due to an extremely

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cavernous foundation, poor abutments, and the great property damage that would be involved in flooding the valleys of North and South Chickamauga creeks. The upper site (mile 471.9) was ruled out in the beginning on the grounds that the bedrock was known to be notoriously cavernous. Later it was deemed wise to explore this upper "Army" site (mile 471.9) in more detail. This later exploration work by the TVA revealed that the top of the rock varies as much as 200 ft and that the bedrock is exceedingly cavernous. In the beginning, the TVA explored a possible site, 0.6 mile downstream (mile 471.3) from the upper "Army" site (mile 471.9), which was on the Chickamauga limestone, but only a few hundred feet downstream from a large overthrust fault. This site, although an improvement over the upper "Army" site, was also quite unsatisfactory due to deep weathering and cavities. The next efforts were to explore two possible sites upstream from the upper "Army" site at miles 472.3 and 474.6. At both of these sites, the top of the rock was found to be very irregular, deeply covered, and cavernous, and therefore unsuitable for a dam foundation. It was then decided to explore a possible site as far downstream as the abutments would allow (mile 471.0) and as far away from the shattered effects of the major thrust faulting as feasible. As expected, this site proved to be the best of the sites explored by either the TVA or the Corps of Engineers and was the site selected. More than 50,000 ft of diamond-drill holes were drilled in the search for this site.

Regional Geology

The rocks underlying the Appalachian Valley consist of a great thickness of disturbed but unaltered limestones, shales, and sandstones of many formations, ranging in age from Lower Cambrian to Middle Carboniferous.[6] The limestones are of many types, but all are dense, hard, and soluble. The sandstone and shales are likewise of many types.

The once flat-lying Paleozoic strata of limestone, shale, and sandstone, which formerly covered twice the area it now occupies, have been compressed as if placed in a gigantic vise. Today, in crossing the valley, one can see many parallel valleys, trending northeast and southwest, which consist of sedimentary rocks of various kinds, repeatedly faulted so that the blocks lay shingled against one another. All of the major faults are of the reverse or overthrust type and dip from 15° to 60° to the southeast. The prevalent dip of the strata is similarly to the southeast, although local northwest dips occur.

In eastern Tennessee, about 45% of the valley area is underlain by soluble limestone and dolomite, but in the vicinity of Chickamauga Dam the Tennessee River flows entirely on limestone and dolomite for more than 30 miles above Chattanooga. Sinkholes dot the surface, and underground drainage is the rule wherever limestone or dolomite occurs. In the vicinity of Chickamauga Dam the valleys and ridges vary in elevation from 630 ft to more than 1,000 ft above sea level.

As is characteristic of the South, the bedrock about Chickamauga Dam is deeply weathered. In many cases, the uplands are covered with a mantle of residual clay and chert to a thickness of as much as 200 ft.

[6] "Geology of Alabama," by G. I. Adams, Charles Butts, L. W. Stephenson, and White Cooke, *Special Report No. 14*, 1926.

Description of the Bedrock

Due to much folding and faulting, and the length of the dam, an unusual thickness and variety of limestone and shale strata were encountered in the excavations for the construction of Chickamauga Dam. This group of mainly thin-bedded, dark colored, hard, soluble limestone, and thin zones and beds of shales, has long been known as the "Chickamauga limestone."[7] This type of limestone includes approximately 2,000 ft of the Upper Ordovician system and several similar, but well-known, formations. Much could be written in describing these formations if space permitted, but only the Trenton and upper Lowville formations are described in any detail since they immediately underlie the principal features. Fig. 50 shows the order, areal distribution, and structural relation of the formations to the dam and surrounding area.

The Trenton formation,[8] the upper part of the Chickamauga limestone, forms the major part of the foundation, immediately underlying practically all of the main structures except the south half of the south earth dam. The Trenton is about 175 ft thick, and is composed of two distinct types of limestone. The upper Trenton, or Cannon limestone, formed the surface rock over most of the area around the lock, spillway, and power house, as shown in Fig. 50, and varied in thickness from 0 to 110 ft. This limestone is dark gray to black in color, fine to medium crystalline, and medium bedded. Cavities up to a vertical and horizontal dimension of 25 ft were sometimes present in this formation, practically all of which was removed by excavation under the lock, spillway, and power house. The lower Trenton, or Hermitage limestone, consists of 65 ft of thick-bedded, impure, mottled limestone and several thin beds of bentonite and shale. Eighteen samples of the limestones of the Trenton formation gave an average compressive strength of 16,900 lb per sq in.

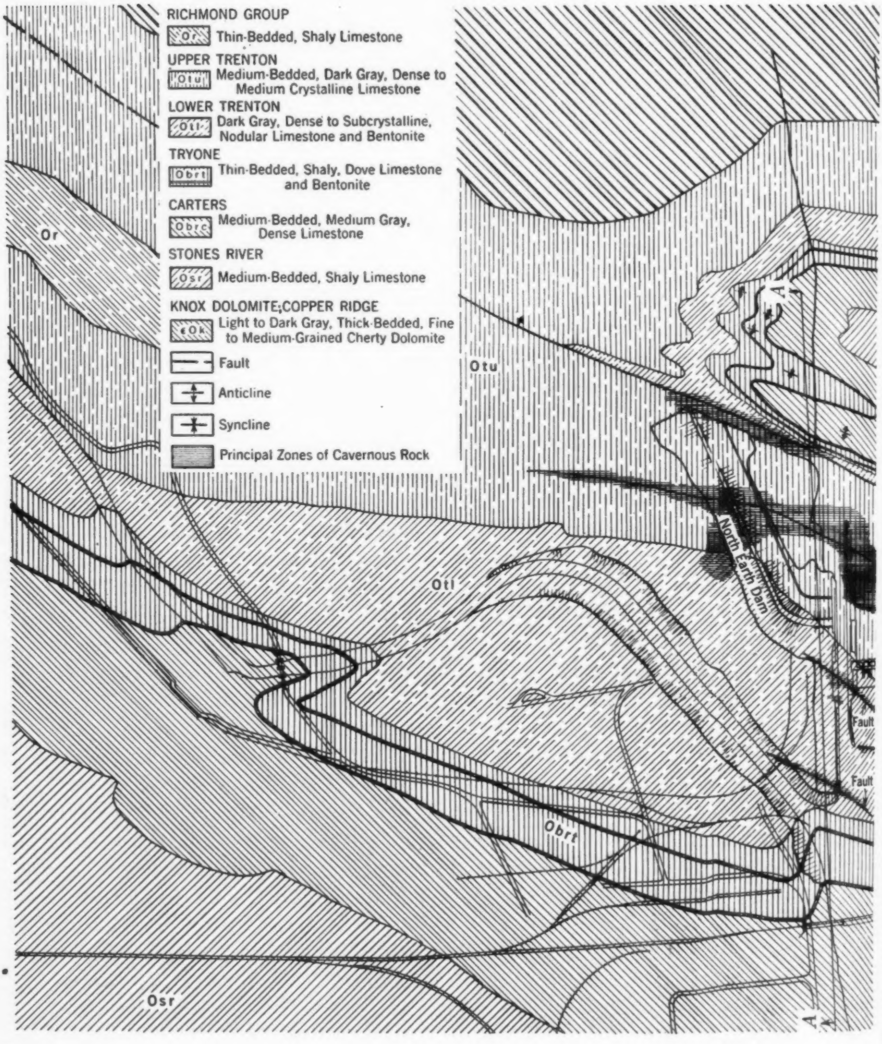
Upper Lowville or Tyrone Formation.—This formation immediately underlies the Trenton and is penetrated by most of the deep drill holes under the spillway, power house, and lock. It also forms most of the rock exposed in the left or south abutment. The Lowville, like the Trenton, is composed of two distinct types of limestone. The lower part has been given the name "Carters" and the upper part "Tyrone" in Tennessee. The lower part is very similar to the underlying rocks and is included with them in the description. The Tyrone consists of 35 ft of thin beds of gray, fine-grained limestone; greenish-gray and red, shaly limestone; and several beds of bentonite. This formation has many distinctive characteristics and has served as a valuable horizon marker in recognizing faults.

The other formations mentioned are described briefly with the south half of the south earth dam, the only place where they occur in the foundation at the surface.

Beds of Volcanic Material.—Interstratified at various intervals with the limestone of the upper Lowville (Tyrone) and the lower Trenton (Hermitage) formations are thirteen beds of soft, white, shaly and sandy clay-like material known to be derived from volcanic ash. This material varies in the size of the individual particles from extremely fine to coarse. The fine, clay-like material

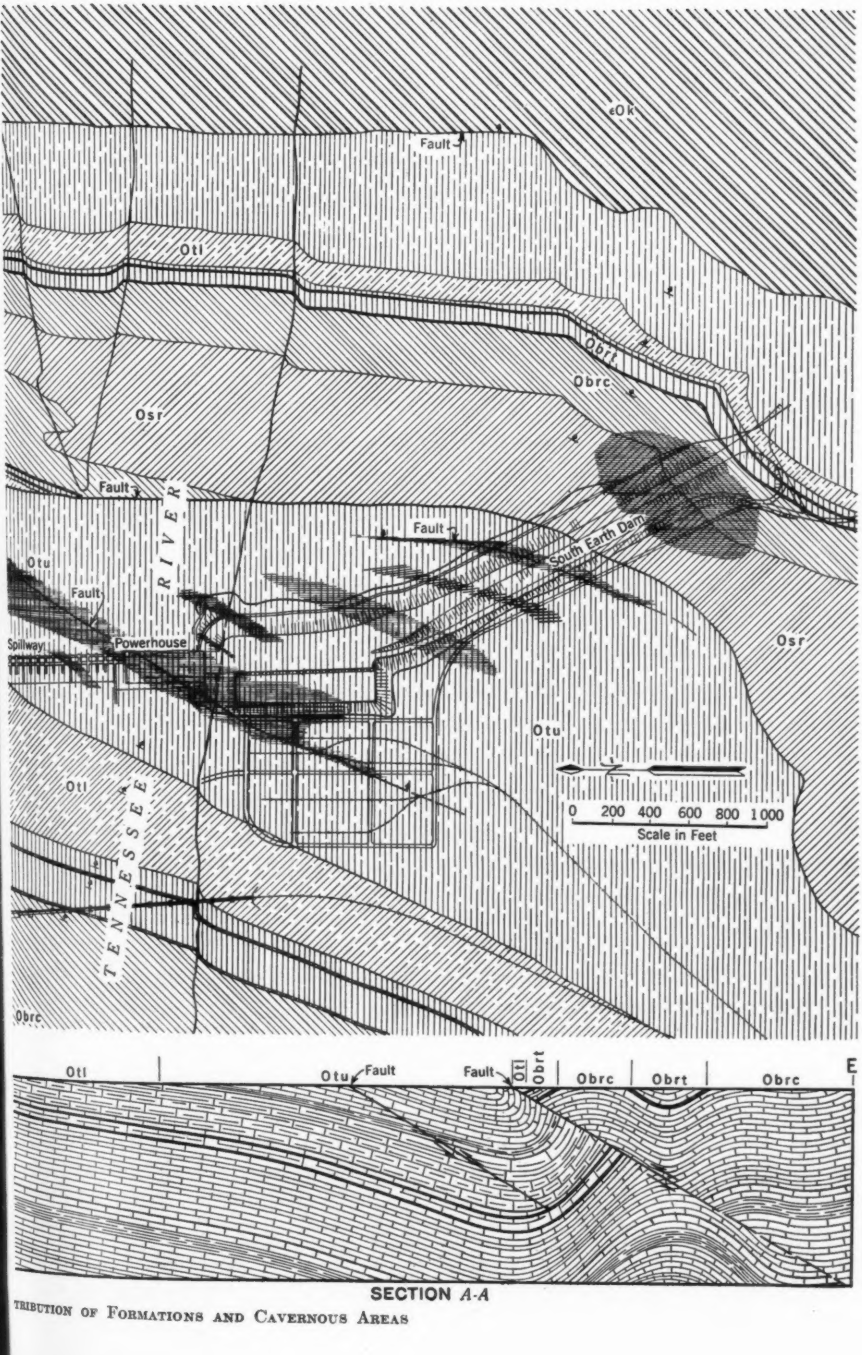
[7] "Chattanooga Folio No. 6," by C. W. Hayes, U. S. Geological Survey, Atlas, 1894.

[8] "Stratigraphy of the Central Basin of Tennessee," by R. S. Bassler, *Tennessee Survey Bulletin 38*, 1932.



SECTION *A-A*

FIG. 50.—GEOLOGIC MAP OF SITE, SHOWING DIS-



SECTION A-A

TRIBUTION OF FORMATIONS AND CAVERNOUS AREAS

is called bentonite. The coarser material is technically classed as arkosic bentonite and bentonitic arkose according to the grain size; but for convenience it is referred to herein as bentonite and shale. The coarser portions of the bentonite beds are porous and friable mixtures of quartz, feldspar, and biotite, and frequently carry a small amount of artesian water. The Tyrone formation contains nine of the thirteen beds of bentonite, two of which have an average thickness of 3 ft. A third bed sometimes attains a similar thickness, but the others are only a few inches or less. The Hermitage contains the remaining four of the thirteen beds, but all of these beds are less than 6 in. thick.

These beds of bentonite are extremely difficult to core satisfactorily with the diamond drill, but it was very important, because of their weak character, to know their location under any structure. The 36-in., or larger core holes, have been extremely helpful in establishing the position of these soft beds.

Mode of Origin of Cavities

Much can be found in literature on the origin and formation of caves in limestone. In most cases the cavities are formed by the percolation of slightly acid ground water through some small primary or secondary weakness or opening in the bedrock. The rate at which any limestone is dissolved depends upon the area exposed to the water and the acidity, temperature, pressure, and rate of circulation of the water. It is evident from the character of the Tennessee River in the vicinity of Chickamauga Dam that it is unable to transport more than silt and sand, except during extreme floods. Therefore, the river bottom is not being eroded mechanically, but it is being lowered slowly by dissolution of the limestones. The long-accepted concept that cavities cannot be developed below the water table has been shown to be quite erroneous by drilling in the Tennessee Valley area.[9] The belief held by some that a limestone under a dam may dissolve and become cavernous in a few months or years is quite erroneous. What usually happens in such cases is that cavities filled with clay or sand have passed unnoticed, or have been allowed to pass without proper treatment, and the material filling the cavity is soon eroded when the reservoir is filled. In considering the rate at which a limestone foundation dissolves, most people forget that the cement in concrete is composed mainly of tricalcium silicates and is less stable and more soluble than most limestones. Most of the larger cavities found at Chickamauga Dam, perhaps, had their beginning many centuries ago.

Description of the Cavities and Weathered Rock

It is not possible to describe in detail all the cavities and weathered rock encountered at Chickamauga Dam, because they have been numerous, but the major difficulties encountered under the main features are outlined as follows.

Right Abutment and North Earth Dam.[10]—Although the right abutment consists of Trenton limestones, intricately folded and faulted and with little cover, only minor cavities and seams were discovered below normal pool level,

[9] "Deep Solution Cavities in the Tennessee Valley Area," by Berlen C. Moneymaker, *Proceedings* for 1937, Geological Soc. of Am., June, 1938, p. 101.

[10] "Elements of Chickamauga Dam," by Lee G. Warren, M. Am. Soc. C. E., *Engineering News-Record*, October 21, 1937, p. 667.

or 682 ft above mean sea level (see Fig. 51). The north earth dam, being 1,365 ft long, stretches across the former North Chickamauga Creek channel and an old flood plain built by the Tennessee River and the creek. This flood-plain material, consisting of gravel, sand, and clay, lay on bedrock or boulders and residual clay at El. 625 and extended as high as 670 ft above mean sea level at a few places. At no place under this entire earth dam did sound bedrock lie immediately below the flood-plain material. Beginning at the east end of the north earth dam, for the first 420 ft along the axis of the dam, high dips, small faults, many small seams and cavities, and a few large cavities, prevailed in the upper 30 ft of the bedrock. In the next 120 ft along the dam axis, the top of the rock was depressed into a **U**-shaped, sinkhole-like depression to a low

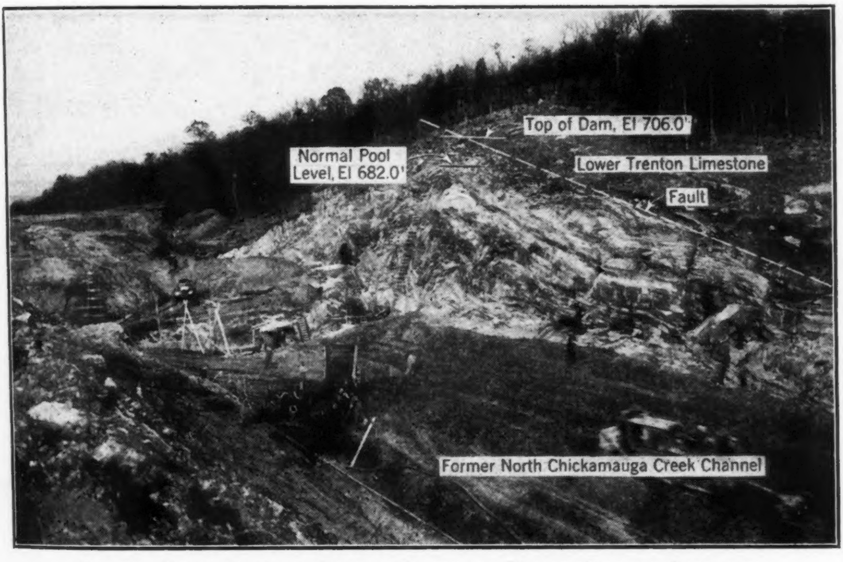


FIG. 51.—NORTH ABUTMENT SHOWING FOLDED AND FAULTED TRENTON LIMESTONE

elevation of 531, or about 90 ft lower than it was either to the east or to the west. This depression in the rock, in its natural condition, was filled with a yellow, residual, plastic clay and chert. For 300 ft west of this depression, also, the bedrock was severely weathered along joints, small faults, and bedding planes. This depression, and the weathered condition west of it, is due to a zone of many small faults, or a fracture zone. West of this point, to the lock, the bedrock lay practically flat. In this latter zone, solution had occurred along nearly vertical joints to a depth of 10 ft, and some large limestone boulders remained over the bedrock.

The Lock.—A depression in the top of the rock of about the same magnitude as the one encountered under the north earth dam was also encountered under the lock piers.[11] The top of the rock under the lock varied in elevation from 625 to 535 ft. Under the downstream guard walls, several small faults,

[11] *Proceedings*, Am. Soc. C. E., June, 1939, Fig. 3, p. 956.

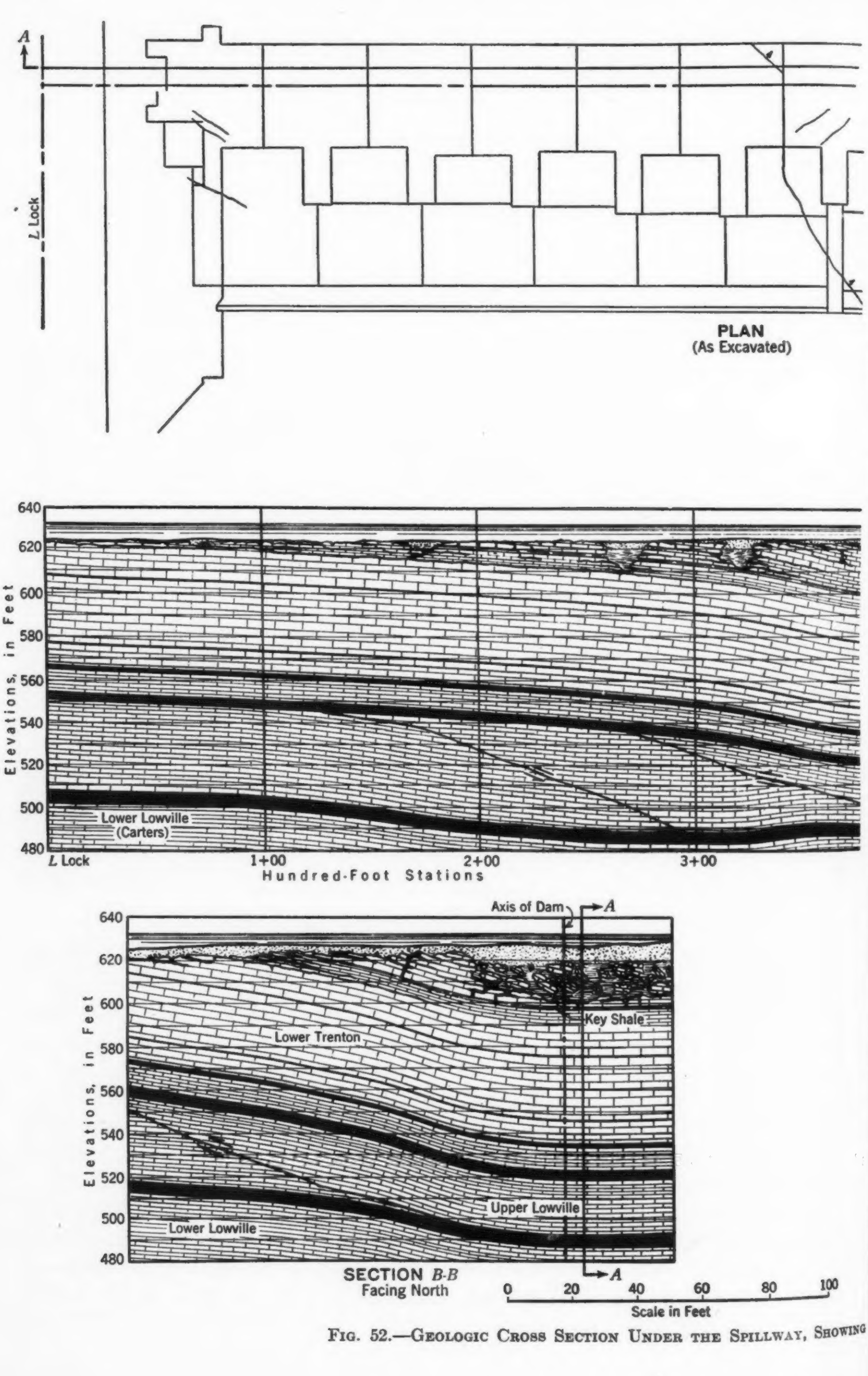
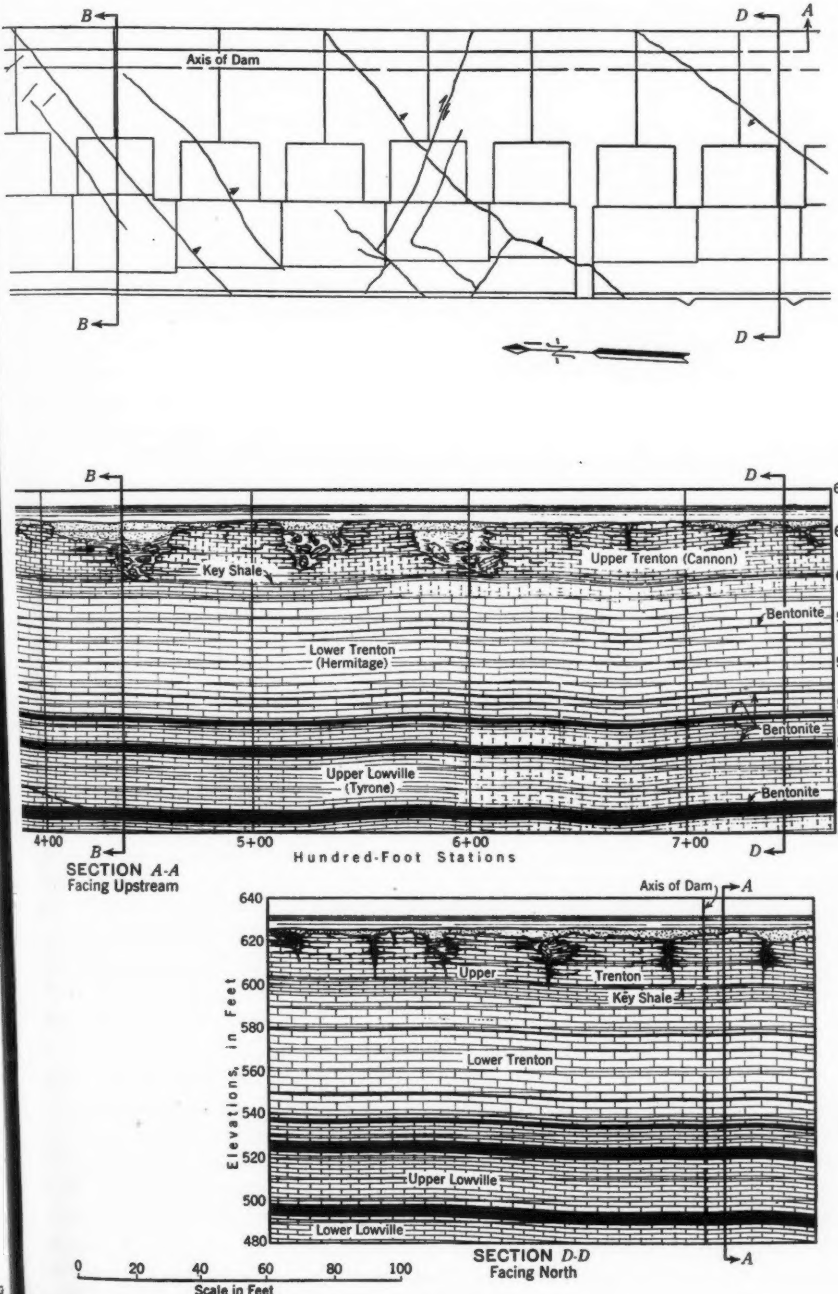


FIG. 52.—GEOLOGIC CROSS SECTION UNDER THE SPILLWAY, SHOWING



WEATHERED AND CAVERNOUS ROCK ABOVE THE "KEY" SHALE

cavities, and seams occurred. Between these two extremes under the lock chamber the bedrock dipped slightly upstream and was only slightly disturbed. Considerable weathered rock, small cavities, and seams occurred above a bed of bentonite which was excavated, but few seams occurred below it.

The Spillway.—The bedrock under the entire spillway dipped upstream from a few degrees to 20°, and the top of the rock varied in elevation from 595 to 625 ft. A bed of shale and bentonite about 1 ft thick, and designated as the "key" bed, outcropped near the lock and dipped to a low elevation of 594 ft at the power house. From the core-drill holes, many geologic cross sections and subsurface contour maps were made of this shale, thus enabling the geologist to predict, within a foot or less, its position throughout the entire spillway and power house. Great gulches, or solution channels having horizontal and vertical dimensions of more than 25 ft, had been developed by solution along strike joints, leaving islands or large weathered blocks of limestone above the key bed of shale and bentonite. This shale and bentonite, although quite thin, formed an impervious layer, and the joints did not extend through it. All the rock was excavated to the bottom of the shale, and not a single weathered seam or cavity was found below it. No part of the foundation of Chickamauga Dam is any better than the foundation of the spillway (see Fig. 52).

The Power House.—In the power-house area the top of the rock varied from 582 to 625 ft in elevation. The foundation of the power house is similar to that of the spillway, but a few notable exceptions occur. The same bed of shale and bentonite occurs progressively deeper until it is repeated by a fault, which has a 10-ft vertical and a 75-ft horizontal displacement. A small cavity persisted along this fault to as low as 560 ft in elevation. In the south half of the power house several large cavities occurred above this fault. In the north end of the power house a mass of limestone boulders and clay extended downward to within a short distance of the key bed of shale. The limestone above the key bed was excavated and only a minor weathered seam remained along the fault in the foundation (see Fig. 53).

The South Earth Dam and the Left Abutment.—This earth dam, approximately 3,000 ft long, crosses the strike of the strata diagonally. The bedrock was covered to a depth of 30 to 40 ft by river sand, clay, and gravel over the north 2,000 ft and by yellow residual clay on the south 1,000 ft of the earth dam. The bedrock immediately underlying the north half of this earth dam is the upper Trenton limestone, described elsewhere. As it occurred at this point, it was folded gently and several faults were present, with several large and deep cavities along joints, bedding planes, and faults (see Fig. 54).

The Trenton limestone is terminated abruptly near the center of the south earth dam by a thrust fault that brings up the lower Chickamauga limestone. A large, irregular cavity roughly followed this fault downward from its outcrop at El. 620 to El. 545. South of the large fault, under the south half of the south earth dam, the beds of limestone dipped steeply into the abutment. The Murfreesboro, Lebanon, Carters, and Tyrone were recognized under this part of the dam, but they were all dark colored, thin bedded, and otherwise quite similar, and for this reason an individual description has been eliminated. Weathering has progressed deeply along these thinly bedded and jointed lime-

stones. At several places clay seams and large cavities were in abundance to a depth of 40 ft below the top of the rock, which required deep excavation.

The bedrock in the left abutment, covered by 40 to 60 ft of residual clay, consists, at the bottom, of a bluish-gray limestone and, at the top, of a series of red and green shaly limestone with several thick beds of bentonite interstratified. This latter series is known as the upper Lowville or Tyrone, described elsewhere. Although the top of the rock in this abutment is slightly below normal pool level, and some shallow filled cavities occur in the bedrock, it is considered satisfactory due to the great thickness of residual clay overlying the rock and the structural advantages that the beds of bentonite afford.

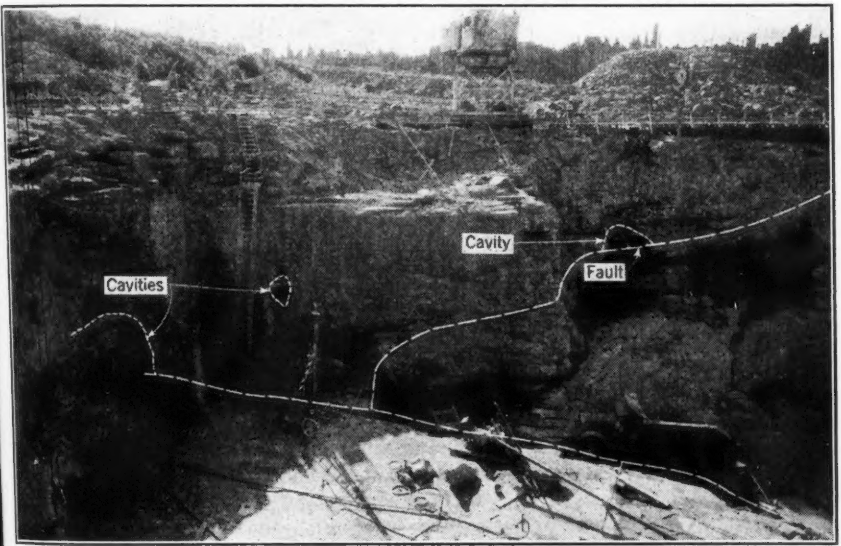


Fig. 53.—Power-House Excavation, Showing Fault and Cavities in Upper Trenton Limestone

Type of Core Drills and Methods Used for Exploration

Practically all small core-drill holes, for purely exploration purposes, have been driven with special equipment operated by hydraulic feed, using a swivel type of double-tube core barrel. Many shot-drill holes ranging in size from 3 to 8 in. have been drilled by several models of shot drills for foundation treatment purposes. Also, a considerable number of large holes, ranging in diameter from 32 to 52 in., have been drilled by equipment described elsewhere.[12]

In exploring a limestone foundation where many soft, weathered seams occur and where many beds of bentonite and shale are interstratified with the limestone, it is imperative that the core recovery be kept as nearly 100% as possible. If the drill water returns, much can be learned by keeping notes on its color, and by comparing with the core losses; but at Chickamauga Dam the drill water was almost invariably lost at the top of the rock or in the first cavity. For exploration purposes under such conditions, the engineers found that a diamond drill

[12] *Proceedings*, Am. Soc. C. E., June, 1939, p. 955.

using an hydraulic feed, a double-core barrel, and a special bit giving a $2\frac{1}{8}$-in. core is by far superior in core recovery to any other type of small core drill. Smaller bits have been used, but the core recovery is much less satisfactory. The small shot drills proved highly unsatisfactory for exploration purposes in coring soft and weathered rock.

The large shot-drill holes of 32 to 52 in., which permit the geologist to enter and inspect the undisturbed rock, is yet unsurpassed by any exploratory drilling method.[13] Where the conditions of the bedrock are as variable as those at Chickamauga Dam, the core from a small drill hole may not represent, even partly, the area immediately around it. The large holes cover 100 times or more the area of the small drill hole and allow one to examine any openings in the walls of the hole for some distance away. The cost per hole for the large hole is usually several times the cost of a small hole, but also the information gained and value of the larger hole are several times that gained from a small hole. A great number of these large holes have been used both for exploration and foundation treatment. It would have been difficult to explore this site thoroughly without their use.[14] On January 1, 1939, 4,618 ft, or 99 of these large holes, ranging in depth from 15 to 90 ft, had been drilled during the construction of this dam.

Functions of the Geologic Staff

Without doubt, the greatest value a geologist can render is in the proper selection of a dam site. Nevertheless, if the site is located on a much folded and faulted limestone such as that at Chickamauga Dam,[15] even after considerable drilling has been done, many "gaps" always remain that can be filled in best after construction has started. For this reason, a two-man geologic staff was maintained at Chickamauga throughout the construction period. The principal aims of the geologic staff were to aid the engineering and construction departments in every way possible in the treatment of the foundation. At many dam sites, the bedrock may be exposed for miles upstream and downstream, and a brief investigation by a consultant geologist is all that is required, but at Chickamauga Dam outcrops of the bedrock in the abutments were of little value in developing the details of the foundation. Most of the bedrock was covered on the flood plains by a 30-ft to 40-ft layer of recent river gravel, sand, and clay, and on most of the abutments by residual clay or older deposits of river material.

For these reasons, it was necessary to depend almost entirely for geologic information on small-diameter shot-drill and diamond-drill cores and on 32-in. to 54-in. calyx drill holes. The aim was to log and record for future use an accurate description of every foot of core recovered and of every 36-in. hole drilled. This may seem to be a small task, and at many dam sites it would be, but at Chickamauga Dam it has been a large part of the work.

It was found necessary to have a very detailed geologic description of the core from every hole in order to make accurate geologic cross sections for

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[13] "Core Drilling for Visual Examination of Foundation Material," by B. M. Jones, M. Am. Soc. C. E., Third World Power Conference and Second Conference on Large Dams (Section A on Dams), September 1–6, 1936; also *Proceedings*, Am. Soc. C. E., June, 1939, p. 953.

[14] "Large-Diameter Core Drill for Geologic Exploration," by Berlen C. Moneymaker and Portland P. Fox, A.I.M.E. *Tech. Pub. 1000*, 1938.

[15] "Bad Rock Limits TVA Dam Location," *Engineering News-Record*, October 21, 1937, p. 665.

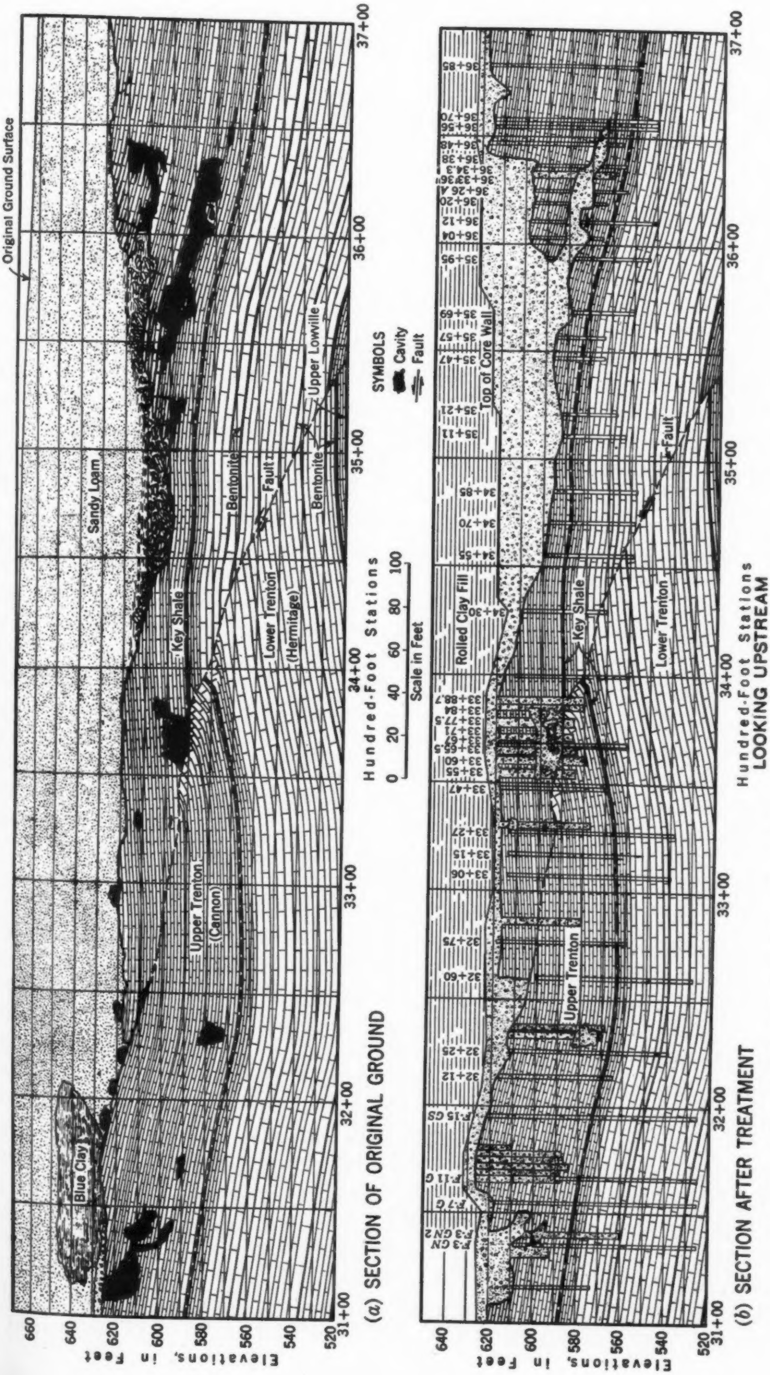

FIG. 54.—GEOLOGIC SECTION UNDER SOUTH EARTH DAM, SHOWING FAULTS, FOLDS, AND CAVERNOUS ROCK

sensible predictions, conclusions, and recommendations. The cores from the holes are logged on a specially prepared form, day by day, as the core is recovered. Four copies of the log are later typed and distributed. It was found desirable to log the core as soon after it was pulled as possible because many things can happen to the core on a construction job to spoil the record.

Method of Interpreting the Cores

During the initial exploration for a dam site at and near Chickamauga Dam, little consideration was given to the detailed geologic structures of the bedrock and detailed description and correlation of characteristic beds and formations. It was early realized that such geologic features held the key to the final solution of the complex foundation problems. Core-drill holes are an expensive means of exploration, and all possible information afforded by them should be obtained. Some engineers are inclined to drill numerous cores without giving sufficient geologic study to the latent information made available by them.

From the beginning of the construction period, the geologists assembled all information in a detailed log of every core-drill hole completed. In these logs are recorded descriptions of the core as to kind of rock, color, texture, thickness of beds, dip, seams, cavities, faults, and any other noticeable physical characteristics. By such detailed method of logging drill cores, it has been possible to depict the geology with a fair degree of accuracy, although it may be buried 30 ft or more by river deposits. By the recognition of the characteristic beds penetrated by the drill holes, it is possible to make natural-scale geologic cross sections and thus to picture the true conditions.

If the beds are not too steeply inclined, and if some definite bed or key horizon can be recognized, it may then be possible to connect this bed or horizon between various drill holes, and in such cases oriented cores are no longer necessary. By this means more than 100 such geologic sections have been made on a scale of 20 ft to the inch and used during the construction of Chickamauga Dam. These sections have been very useful during construction and will form a valuable future record of the foundation.

Spacing and Depth of the Exploration Holes

For preliminary exploration purposes the initial spacing of the diamond-drill holes was on 100-ft centers and reduced to 50 ft at a few places when necessary. With the 100-ft spacing in a cavernous limestone foundation, it is possible to show only very roughly, on a graphic section, the top of the rock and a skeleton of the geologic structure. Holes spaced this far apart in a cavernous limestone cannot represent the conditions faithfully. It was discovered later that some of the largest cavities and worst conditions fell between the holes on the 100-ft spacing. In a few cases the holes had missed large cavities by only a few inches. Several of the small, and what appeared to be unimportant, cavities and seams in the wide-spaced holes were actually the small ends, or on the edge, of some large cavities. With holes on 50-ft centers, the geologic structure, if not too complicated, can be shown fairly accurately on a graphic section, but the cavities cannot be so shown. With holes on 25-ft centers all of the details of the rock structure can be predicted accurately and a good idea of the size and distribution of the cavities can be had; but if the bedrock

extremely cavernous, holes spaced on 12.5-ft centers will be of considerable aid in planning the methods of treatment. These closer holes have been drilled on some one definite cutoff line and where they can be later used for grout holes. The advantages of placing all the holes along one line rather than spacing the same number of holes over a large area is that the geology can be traced in detail and the good and poor sections can be projected with a fair degree of accuracy which otherwise would be impossible.

Only a limited number of the diamond-drill holes have been driven on an angle from the vertical or horizontal. Where cavities occur along vertical joints in horizontally bedded limestone, angle holes can be used with greater value than vertical ones; but at Chickamauga Dam, where nearly horizontal beds occurred, cavities were shallow and few needs for angle holes arose.

The depth of the exploration holes was varied between 50 and 200 ft in order to encounter faults, beds of bentonite, and zones of deep weathering. On January 1, 1939, 58,244 ft of diamond and 47,772 ft of small shot-drill core had been drilled and carefully logged by the geologic staff during the construction of the dam.

Geophysical Method of Exploration

Three attempts were made to explore the conditions of and the depth to bedrock by electrical resistivity measurements. Considerable data were obtained by the patented tester, and the great difficulty was not with operation of the tester itself, but in the interpretations of the data obtained. Perhaps the main difficulties in interpretation of the data were due to an overburden of variable character and quite variable conditions in the bedrock which were not indicated by the widely spaced drill holes. This method could be used best before drilling so that no drill casing or water lines would be in the way to affect the resistivity measurements; and if any unusual conditions were noted they could be checked with drill holes at the beginning. By this means it would be possible to investigate the worst conditions of a proposed dam site first and not after many drill holes had been driven.

Source of Information and Acknowledgments

The writer was connected with the geologic staff at Chickamauga Dam from the construction in 1936, and most of the information presented herein has been obtained first-hand from his own observations and with the assistance of Leland F. Grant and John C. Dunlap. The writer is also indebted greatly to many engineers on the engineering and construction staffs at Chickamauga Dam. Most of the preliminary geologic exploration was in the care of Harry N. Eaton, Robert M. Ross, Roy Caldwell, and William Lovelace.

Conclusions

The exploration and geologic studies at Chickamauga Dam clearly revealed the structure, character of the bedrock, and the extent, relation, and depth of the cavities and weathered rock. The main problems at Chickamauga were related to the foundation, and geologic studies were of material aid in overcoming difficulties in the construction of the dam. Fortunately only a few dam foundations are as complicated as this one, but in such cases only painstaking detailed geologic studies can satisfactorily unravel the real nature of the foundation problems.

FOUNDATION TREATMENT AT CHICKAMAUGA DAM

By James B. Hays,[16] M. Am. Soc. C. E.

Synopsis

Chickamauga Dam is on the Tennessee River, 7 miles above the City of Chattanooga, Tenn. The river flows westerly at this point. The project consisted of a lock adjacent to the north, or right, bank; a spillway across the main part of the stream; and a power house on the south, or left, bank, with two sections of earth dam, one on the right flood plain 1,300 ft long, and another on the left plain 2,900 ft long, known as the north and south earth dams, respectively, making a total of 5,900 ft. In plan, the structure is a flat U shape. The axis was curved downstream to follow the best foundation rock available. Fig. 55 shows the general plan of the dam.

Due to the nature of the foundation rock, with its numerous channels, cavities, seams, faults, and folds, the problem of making a watertight cutoff was one of greatest importance. Perhaps at no other dams were more thorough investigations made. Grouting covered a wide range, using different materials and methods, and it is felt that, due to the extensive nature of the work involved, the information and experience obtained on the project will prove of value to others.

The cutoff was grouted, using only cement and water. Other grouting for temporary construction purposes, and to fill cavities under the earth dams, was done with other materials and mixtures.

Introduction

Description of Sequence of Operations.—Construction began in January, 1936, the work in the river being done in three stages. The construction of the lock, at the right bank, was the first stage and was completed, ready for temporary operation, by June, 1937. The lock chamber is 60 ft wide by 360 ft long and provides for a lift of 50 ft. The second stage followed and included the construction of 14.5, of a total of 18, bays of the spillway. Each bay consisted of a 40-ft gate opening and an 8-ft pier. This stage was completed in the winter of 1937–1938. Stage three included the remainder of the spillway and the power house. This was started during the spring of 1938. During the third stage, the river was diverted through the spillway. During the operation of the foregoing program, the earth-dam sections and the diversion channel for North Chickamauga Creek, which had to be diverted around the dam, were under construction. Twelve bays of the spillway in the second

[16] Constr. Engr., Kentucky Dam, TVA, Gilbertsville, Ky.

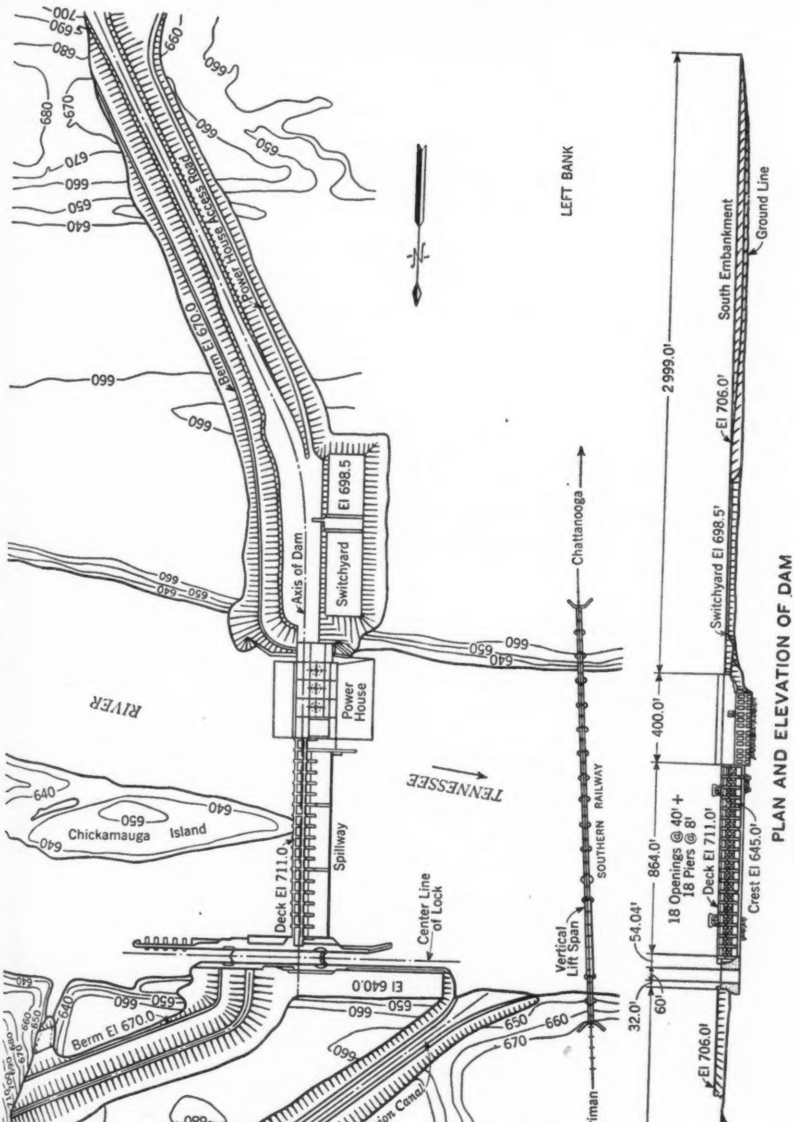

FIG. 55.—PLAN AND ELEVATION OF DAM

stage had been left 13 ft below final crest elevation to reduce the head on the upstream cofferdam of stage three. The normal reservoir pool elevation is 682; the tailwater normal elevation, 632; and the top of the earth fill, 706. The top of the lock walls is 692 and of the spillway and intake, 711. The level of the flood plains was between 660 and 670. The river bed varied from 625 to 628, all elevations above sea level. The earth dams were rolled fill and had side slopes of 1 on 3 with rock riprap over a gravel or crushed rock filter blanket.

When construction was started on the project in January, 1936, exploration was still under way. The axis of the dam had been selected after considerable study of several sites. Along the axis, exploratory holes were placed, as a rule, on 100-ft centers with additional holes in certain areas. The closer spaced holes were mostly in the river where the concrete structure was to be founded directly on rock. Elsewhere in this Symposium, Mr. Fox describes the thorough study made in conjunction with the foundation treatment.

The method of making a cutoff under the earth-dam sections was not decided until July, 1936. The average depth of the overburden was 40 ft. It was first suggested that either grouting alone or a sheet-pile cutoff, or a combination of the two, would suffice as against an open-cut trench to sound rock; and, in order to determine the proper method, an experiment was conducted. An area on the left bank, along the axis of the dam and close to the river, was selected for the test. Exploration holes had indicated cavities in this area. A square pattern of 16 holes, drilled in four rows, spaced on 10-ft centers each way, was drilled through the overburden and 100 ft into rock. These holes were grouted, taking a total of 15,000 cu ft of cement. A trench 200 ft long was excavated along the axis with the aforementioned pattern of holes at the north end. The base width was 30 ft and side sloped 1 on 1.5. The first 20 ft was a river alluvium of sandy clay. The bottom portion graded from sand and gravel to large, loose boulders or slabs of limestone. When sound rock had been uncovered, a 36-in. hole was drilled at the north end where the original grouting had been done in cavernous limestone. The 36-in. hole at the north end went through a cavity containing grout and gravel. The leakage was small (approximately 6 gal per min), although the hole was only 150 ft from the river. At the south end of the trench, a pattern of holes similar to the first set, but only 40 ft deep, was drilled and grouted. This pattern required 2,400 cu ft of cement. Fig. 56 is a view of the bottom completely cleaned up; Fig. 57 shows the general plan of the operation.

As a result of the experience, and based on the condition of the rock and the overburden, it was decided that a sheet-pile cutoff would be of no value, since a good tie to a sound rock was not possible, due to the mass of boulders. Grouting alone would be very expensive and still uncertain. It was decided that for the cutoff under the earth-dam sections an open cut should be made to sound rock, to be followed by drilling and grouting 40-ft holes on 12-in. centers. These holes were drilled first on 4-ft centers, and, after washing, grouted. These holes were also of assistance in locating cavities. Those cavities relatively close to the surface were opened and backfilled with concrete along the line of the cutoff. Deep cavities were treated by intercepting them

FIG. 56.—PLAN AND ELEVATION OF DAM

with 36-in. drill holes to be used as shafts for cleaning out the cavity and backfilling with concrete. Following this operation, grout holes were drilled midway between those on 4-ft centers. These were washed and grouted. A third set of holes was drilled, washed, and grouted in alternate spaces and, as a final check, the remaining spaces were treated similarly, thus finally spacing the holes on 12-in. centers. Since the bits used on the drill steel averaged about 2.5 in., the rock not actually drilled in the final operation was only about 9.5 in. out of each foot. The system of drilling closely spaced holes on a single line reduced the hazard of missing small cavities which might be the case if holes were staggered over a wide zone. On completion of the grouting, a concrete wall located on the center line and over the grout curtain, 5 ft high with batters of 5 on 1, was built and the trench backfilled with suitable material. Fig. 58 shows the trench ready for backfill.

No further work was done on the south side, or left, bank until the spring of 1937. Since the lock was the first stage of the operations in the river, the next work was done on the earth embankment on the north bank, following the foundation treatment for the lock.

Lock

The grouting program adopted for all of the concrete structures was in three stages. First, there was a general consolidation grouting (an average of 30 ft deep) of the entire area to be covered, after all loose, cavernous, and seamy rock was removed. The cutoff grout curtain followed, with closely spaced holes to a depth of 40 ft. After concrete had been placed, deep holes along the line of the cutoff curtain were drilled and grouted.

The consolidation system was started with a regular pattern of holes laid out in rows 10 ft on centers and the holes spaced 10 ft in each row. Additional holes were drilled and grouted until all areas were made tight. Re-drilling and grouting was repeated as much as five times in certain areas. Pressures were limited to 30 lb per sq in. Uplift gages were used wherever necessary and further controlled the pressures used. The consolidation grouting required 41,400 lin ft of drilling and consumed 19,300 cu ft of cement.

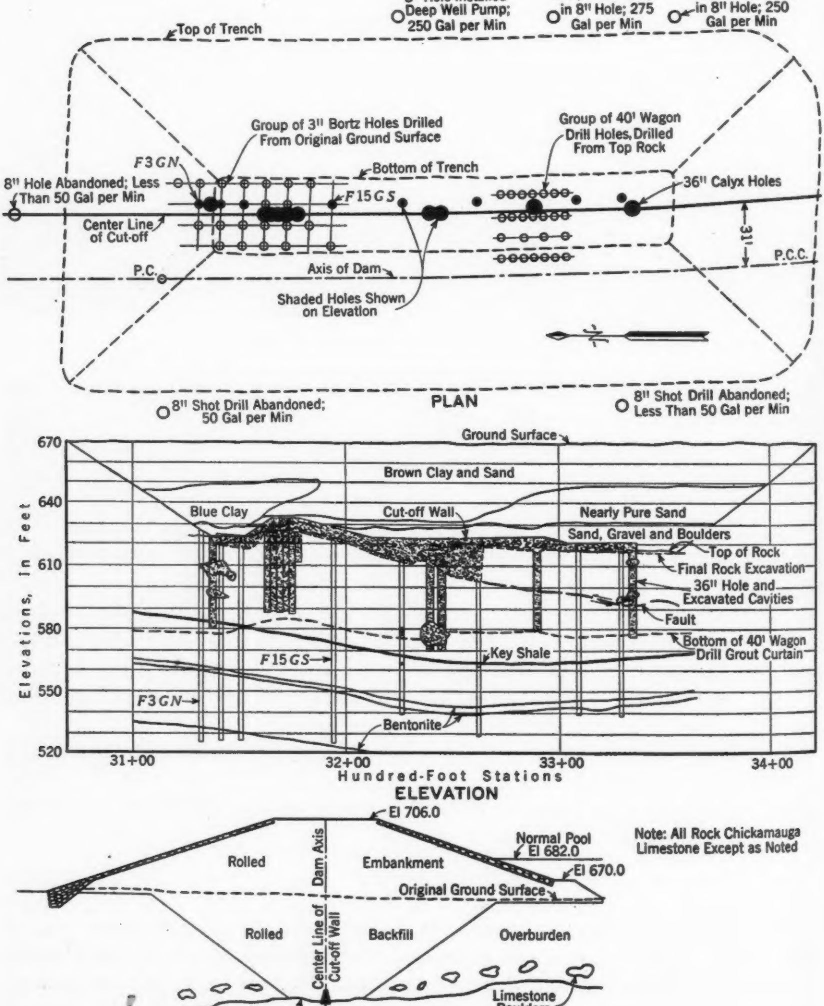

FIG. 57.—EXPERIMENTAL TRENCH TRIAL TREATMENT

The cutoff curtain under the lock followed and completely surrounded the lock chamber, tying in with the spillway and the north earth dam. These holes were drilled 40 ft deep on 40-ft centers and angled about 20° from the vertical and in the plane of the cutoff. A second set was drilled on the same spacing and slope but angled in the opposite direction. All were washed before grouting. The reason for the different system from that adopted for the cutoff under the earth embankment sections was due to the fact that requirements for sound rock on which to build a concrete structure were more exacting than for the earth dams; hence the close drilling was not necessary. The angle holes would intercept vertical as well as dipping joints, whereas, under the earth dams, the holes were so closely spaced that connections with such joints were generally assured.

FIG. 58.—EXPERIMENTAL TRENCH, FACING SOUTH; READY FOR BACKFILL

In the cutoff grouting, pressures were limited to 50 lb per sq in., and uplift gages were not used. Theoretically, the grouting was being done on a rather narrow band and a proportionately larger volume of rock could be considered as resisting the grout pressures compared to consolidation grouting that was done over a wide area at one time. The consolidation grouting had filled all of the larger and more extensive seams. The cutoff grout curtain required 10,800 lin ft of drilling and took 820 cu ft of cement.

Following the cutoff grouting, a row of diamond-drill holes 60 ft deep was drilled, spaced 20 ft, center to center, along the land wall of the lock from the north earth dam tie-in across the upper miter sill and along the river wall of the lock to the spillway. These holes were drilled and grouted alternately. The total of 69 holes required 1,957 cu ft of cement. To check the secondary holes that required it, a third set was drilled and grouted. However, these

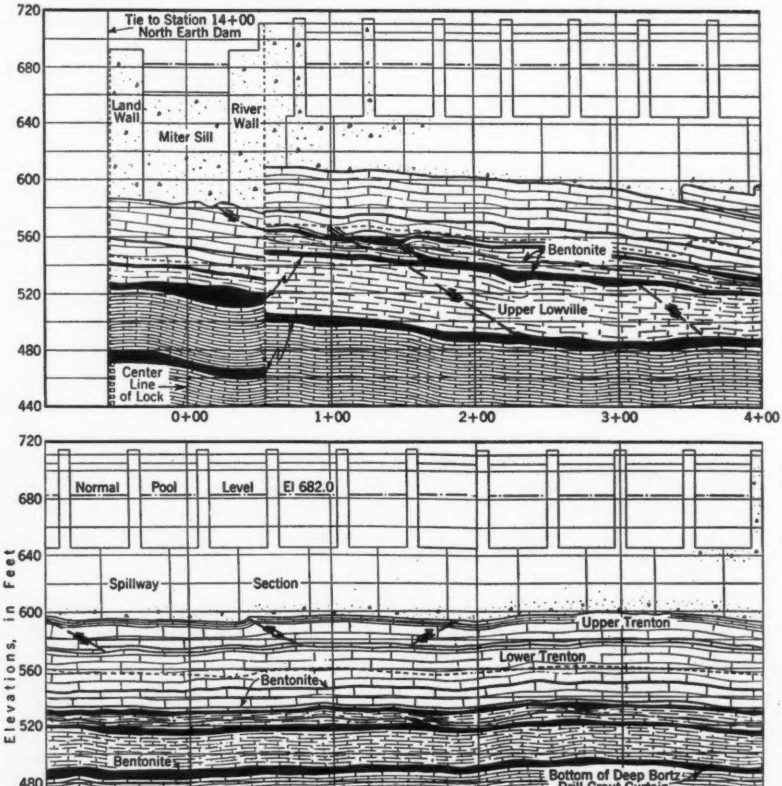

Fig. 59.—Profile of Power House, Spillway, and Miter Sill

latter holes were drilled from 100 to 110 ft deep in order to reach the thick beds of bentonite which carried a small quantity of artesian water. A total footage of 550 lin ft was drilled and 572 cu ft of cement were used. Fig. 59 shows a general outline of the cutoff grouting across the lock, spillway, and power house.

North Earth Dam.—Excavation of the cutoff trench was started at the land wall of the lock, where the conditions were different, at first, from those found in the experimental trench on the opposite bank of the river. For a distance of 500 ft from the lock wall, few loose boulders were found. The rock surface was comparatively level but was crisscrossed with vertical solution channels that followed the dip and strike joint systems in the rock. The width varied from a few inches to 3 or 4 ft, and the depth from a few inches to 8 or 12 ft.

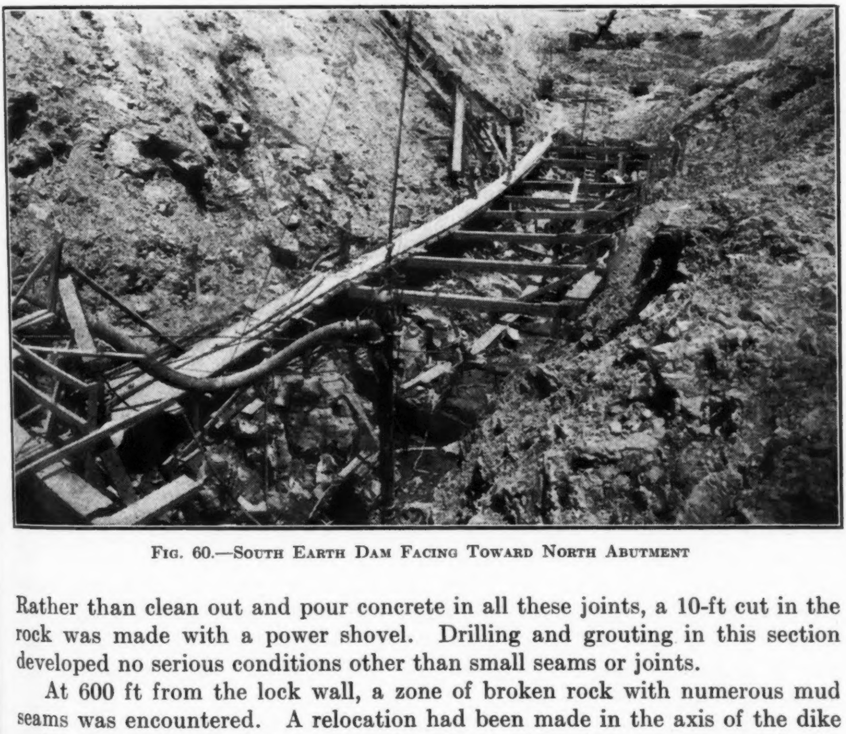

Fig. 60.—South Earth Dam Facing Toward North Abutment

Rather than clean out and pour concrete in all these joints, a 10-ft cut in the rock was made with a power shovel. Drilling and grouting in this section developed no serious conditions other than small seams or joints.

At 600 ft from the lock wall, a zone of broken rock with numerous mud seams was encountered. A relocation had been made in the axis of the dike and cutoff to avoid a known area where the rock surface was at great depth. This change brought the axis along a line on which, for a short distance, no holes had been drilled directly, although there were a number of them located a short distance on either side.

In the bottom of the trench the rock had been excavated to the point where bracing and cribbing would have been necessary to remove all of the seamy rock, and it was thought that by repeated drilling, washing, and grouting, the zone could be made tight. This attempt was not successful. Fig. 60 shows

this section of the rock. The bottom of the trench in the rock averaged 10 ft in width. It was decided to cover the bottom with a mat of concrete, build the low cutoff wall, partly backfill the trench with clay to increase the stability of the sides of the cut, and then drill 36-in. holes through the mat and excavate the rock by tunneling where the mud seams existed. This operation covered a length of 100 ft along the axis of the cutoff with a general depth of 40 ft. After the tunnel work had been completed, it was backfilled with concrete and grouted. Diamond-drill holes were drilled through this zone and grouted as a final check.

The next 75 ft along the axis was fair rock. Many small seams were found along joints, bedding planes, and two minor faults. In the next 150 ft, a deep depression 65 ft below the normal rock surface was found. Holes close by on either side of the axis gave no indication of its presence. The open-cut trench had been made of a width satisfactory for rock of normal depth. Exploration holes were put down across this zone. These indicated that the depression was filled with a tight clay with zones or pockets of dense chert. Except for clay-filled seams at either end, the remainder of the rock was sound and free from seams or openings. Rather than widen the open trench (which would require the excavation of a very large amount of earth), or excavate and shore a narrow trench to rock, it was decided that a sheet-pile cutoff across this area, followed by grouting of the rock and the contact between the rock and clay, would be satisfactory. At each end, a 36-in. hole was drilled to intercept whatever clay seams existed. These were cleaned out and backfilled with concrete.

From the end of the deep area to the base of the right, or north, abutment, a distance of 340 ft, the rock was excavated to various depths in following mud-filled or clay-filled seams, as they were found. Three 36-in. holes were required to treat deep cavities. The right abutment, which was on the opposite side of North Chickamauga Creek from the dam, was an exposed face of rock that had been folded to a high degree. It was necessary to remove an average of 10 ft of rock to reach material sufficiently sound to grout. From the base of the abutment to a point 350 ft beyond the end of the dam, a total distance of 425 ft, holes from 100 to 175 ft deep had been drilled and grouted on 50-ft centers, followed by holes midway between. In addition, holes penetrating the same zone at a 45° angle were drilled and grouted, using the same spacing and sequence. The bottom of these angle holes covered a zone 350 ft from the base of the abutment. No grouting was done above the elevation of the top of the dam.

In drilling these holes, a fault was found, outcropping at the top of the dam and dipping downward about 30° below horizontal into the abutment. Below the fault, the rock was sound; above it, there were many seams, down to the thick beds of bentonite. Since the outcrop of the fault, as well as the beds of bentonite, was higher beyond the end of the dam, there was no danger of leakage around the end of the abutment.

In addition to the concrete wall normally used, a steel sheet-pile cutoff was built at the abutment. The sheet piling was extended horizontally a distance of 50 ft into the fill from the rock. A steel sheet-pile cutoff, similar

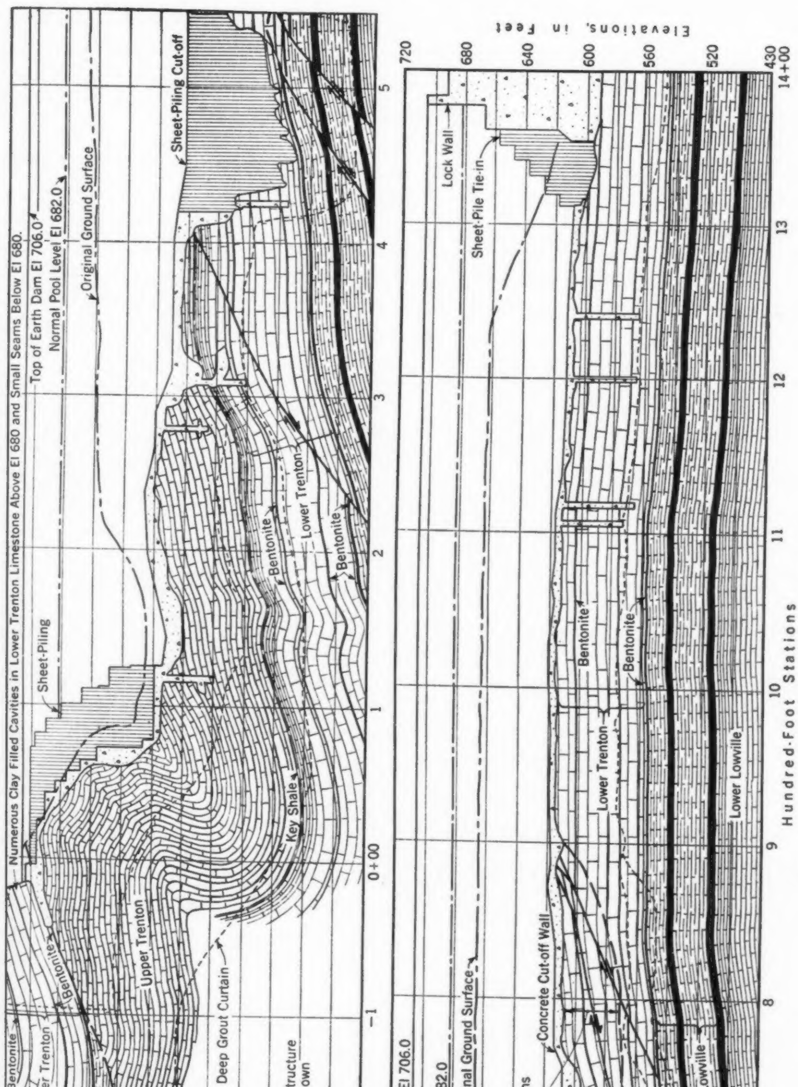

FIG. 61.—PROFILE OF NORTH EARTH DAM

to that in the right abutment, was used at the junction of the north earth dam and the lock wall. The total quantities in the foundation treatment under the north earth dam (see Fig. 61) are: Wagon drilling, 45,000 lin ft; diamond drilling, 6,400 lin ft; 36-in. diamond-drill holes, 500 lin ft; and cement, 31,200 cu ft.

South Earth Dam.—The foundation treatment on the south earth dam was resumed in the early spring of 1937. A section 500 ft long, located immediately south of the experimental trench, was opened. The original exploration holes, spaced on 100-ft centers, did not indicate anything unusually serious.

However, by the time the aforementioned trench had been dug to about 75% of the estimated depth, it was found that the problem of leakage water was to be a major one. An attempt was made to drill and grout along the upstream side of the trench in order to intercept the leaks. Drilling was done with 6-in. shot drills in order to have a fairly large hole through which to force grouting materials. Diamond drills were operated in the bottom of the trench to locate sound rock more accurately between the original 100-ft holes. Excavation of the trench was continued. The flow of water increased with the depth of excavation and more pumps were installed. As the work on the 6-in. holes on the bank continued, the geology was developed. Some of the holes were in solid rock. However, the underground channel was located and attempts made to grout it. Asphalt was tried, but the process was so slow and the cavities so large, due to removal of material by pumping, that this and other grouting was abandoned. Finally, the water courses in the rock were located and two of them were tapped by 36-in. holes drilled in the trench near the upstream side. Large pumps were placed in these holes and, in addition, two other large pumps in the bottom of the trench enabled the crews to complete the excavation. The total volume of water pumped reached a maximum of 15,000 gal per min. Several 36-in. holes were drilled to deep cavities. All of the foregoing trouble occurred in the north 250 ft of the section (see Fig. 62). This end was nearest the river and immediately south of the experimental trench. The geology of the area upstream from the section indicated a low trough in the rock surface along the direction of the strike. An overhanging, cliff-like structure on the up-river side of this low area had two systems of cavities that were intercepted by the trench and were fed directly from the river.

As a result of the foregoing experience, additional holes were drilled on 25-ft centers, prior to excavation, on the center line of the cutoff for the remainder of the dam. This meant the drilling of three holes between each of the original exploration holes which were on 100-ft centers. A large number of new cavities were developed, many of which would have been costly to handle had they not been located in advance.

As a precautionary measure, all exploration holes on or near the center line of the cutoff were grouted. Many of the holes were treated in more than one stage. Packers were set to grout the bottom solid rock sections first and were followed by grouting seams and cavities above, individually or in groups, as the conditions required.

In order to block off, in advance of construction, any water leakage into the trench or cavities, two rows of holes were drilled parallel to the center line, one upstream about 40 ft and another row the same distance downstream. The angle of strike of the rock was taken into consideration in locating these side holes, since the cavities were generally in the same direction. Without washing, these holes were grouted with a low-cost "prescription" grout in which sand, bentonite, and cement were used. The ratios varied somewhat at first but usually 4 parts of sand, 1 part bentonite, and from 2 to 3 parts of cement were used. This mixture cost about half that of straight portland cement grout and was considered sufficiently effective for the purpose. The pressures used were generally 0.5 lb per sq in. for each foot of overburden, or, if in deep rock, 1 lb per sq in. was added for each foot of depth in the rock.

Fig. 62.—Cutoff Trench, South Earth Dam, at Point of Deepest Cavity

The bentonite was mostly from Wyoming or the Dakotas. The most suitable grading was: Not less than 90% through 50 mesh; not more than 50% through 100 mesh; and not more than 20% through 200 mesh. This grading was readily mixed, had high swelling powers, and made a good gel.

Bentonite has a dispersion action on the sand and cement, thus keeping all of the materials in suspension. Without the bentonite, the sand would have washed out of the mix and would not have been effective in stopping water. The material is slow-setting and reaches a strength considerably lower than cement grout. The material, when set, is tough and resists erosion to a fair degree. When allowed to dry out, it shrinks and cracks. The bentonite must be mixed with the water before the cement is added; otherwise the dispersing action is mostly lost. The material has a slippery feel until the

final set. Another advantage in using bentonite is that it reduces the wear on grout equipment, particularly pumps, when sand is used.

The aforementioned method of preliminary grouting of cavities and solution channels, prior to excavation, was successful in preventing leaks into the excavation. One of the cavities (see Mr. Fox's paper, heading "Description of the Cavities and Weathered Rock: The South Earth Dam and the Left Abutment") extended down to an elevation 90 ft below the river surface and leakage was very small.

In the last 900 lin ft of the cutoff for the south earth dam, no side grouting was done. Throughout this section, all of the seams and cavities were found to be filled with a residual clay. They were not refilled solution channels as were those under the remainder of the structure.

Throughout the south earth dam, the conditions could be stated, in general, as follows (see paper by Mr. Fox for greater detail): In the first 2,000 lin ft south of the river, there were three zones varying from 200 to 400 ft in length that had large cavities and solution channels. The last 1,000 ft, up to the south abutment, had numerous cavities, seams, faults, and joints. Through most of this section the weathered and cavernous rock was excavated from 20 to 40 ft below normal rock surface. The 500-ft section nearest the abutment had to be shored.

The total quantities of cement grout used in the south earth dam were 16,540 cu ft in exploration holes and 70,900 cu ft in the cutoff. In addition, 158,614 cu ft of "prescription" low-cost grout was used, the materials involved being as follows:

Material	Cubic feet
Sand	72,133
Bentonite	14,243
Cement	90,000

In a few holes, sawdust and shavings were added, which totaled 2,238 cu ft, loose measure. A total of 1,500 bbl of asphalt and pitch were used as mentioned previously.

Because of the fact that many, more or less open, cavities were encountered in the cutoff trench under the south earth dam, it was considered advisable to fill them to prevent possible future sinkholes in the rolled fill. Undoubtedly, these cavities were open under adjacent parts of the earth fill. The proposal to fill these cavities with a clay slurry was objected to on the grounds that no pressure could be applied to remove the surplus water from the mixture and thus solidify it. As a result of a series of laboratory tests, F. H. Kellogg, associate materials engineer, found that a local river terrace deposit of red clay could be stabilized to form a satisfactory grout by the addition of portland cement. This clay had a chemical analysis (based on samples dried at 110° C) as follows:

Compound	Percentages
Silica (SiO_2)	62.83
Alumina (Al_2O_3)	18.26
Iron oxide (Fe_2O_3)	7.50
Lime (CaO)	None
Magnesia (MgO)	0.28

Compound	Percentages
Sulfuric anhydride	None
Phosphorus pentoxide (F_2O_5)	0.18
Titanium dioxide (TiO_2)	1.45
Manganous oxide (MnO)	0.028
Sulfide sulfur, as FeS_2	None
Loss on ignition, 110° C–1,000° C	7.16
Insoluble residue	82.68
Total sulfur (S)	None
Potassium oxide (K_2O)	0.61
Sodium oxide (Na_2O)	0.02

The sieve analysis was 25% clay (− 0.005 mm), 47% silt (+ 0.005 mm and − 0.05 mm), and 28% sand (+ 0.05 mm). This clay is of the type that crumbles on drying, which makes it suitable for mixing and handling in most types of grout equipment. This stabilized clay grout is mixed in the proportion of 7 cu ft of clay and 1.5 cu ft of cement. The total water amounts to 6 cu ft, including that already in the clay. This material reaches an initial set in 11.5 hr and a final set in five days. Tests show the shrinkage to be practically nothing. The shear strength is 34.8 lb per sq in. Cored samples of clay grout have been obtained with both shot and diamond drills, indicating that the material has hardened satisfactorily.

Two field tests were made which further indicated that this grout would be satisfactory. Two 50-gal drums were filled with water. One was also filled with crushed rock, 1.5 in. to 3 in. in size. A grout pipe was inserted so that the grout was introduced at the bottom and the water was displaced from the bottom upward. When all of the water had been displaced, the grout pipe was removed and the material allowed to set. The remaining drum was filled with grout by allowing the mixture to drop through the water until the drum was full of grout. The object was to determine whether the cement would separate from the mixture under such conditions. Practically all of the open cavities had water in them and it was important to know about any possible separation of cement from the clay.

Both of these samples were allowed to set for one week, when the drums were out and the cylinders of grout removed. In both cases the grout was hard, except for a few inches at the top. The samples were left exposed to the atmosphere, heavy rains, and long dry spells and, after six months, they were still in excellent condition until freezing weather, when the sample that had been poured directly in the tank of water began to spall. These samples were deposited under no pressure. Under regular grouting operations where pressure is applied, any surplus water is forced out, thus further solidifying the mix.

This material was selected for filling cavities under the earth dam not otherwise treated in the cutoff. By observation in the cutoff trench, and from exploration holes, the trend of the cavities was established. Many seams and openings existed in the rock side walls of the core trench which could not be filled with clay in the backfilling operation.

In all such cases, grout pipes were inserted and brought up as the fill progressed. As soon as the fill reached the natural ground line, these pipes were grouted with the stabilized clay grout.

In the section where there had been heavy pumping, it was necessary to place large casings, for pump sumps, as the fill progressed. Corrugated culvert pipe was used for casing. When the fill had been completed to a point above the ground-water table, that was the same as the river surface, the pumps were removed. Grout pipes were installed, reaching to the bottom. The casings were filled nearly to the top with coarse rock. A concrete cap, 1 ft thick, was placed over the rock in each casing. Two vent pipes extended through each cap. The casings were filled with clay grout.

In addition to filling cavities under the earth dam, the area upstream from the section where the heavy pumping was required was filled with clay grout. This was done for two reasons. One was to eliminate any settlement immediately upstream from the toe of the dam and the other was to prevent leakage around the land connection of the upstream area of the third-stage cofferdam. In this area, a total of 36,000 cu ft of clay grout was used, in addition to 37,996 cu ft of sand-bentonite-cement grout.

FIG. 63.—SPILLWAY EXCAVATION

Spillway and Power House.—The excavation of the spillway and part of the power-house sections was greatly simplified by the occurrence of the shale and bentonite which was the key bed used in tracing the geologic structure over most of this area. Above this bed were numerous solution channels and cavities. Where this bed was covered with a reasonable layer of rock and where no faults or sharp folds existed, it was the rule rather than the exception that there were no cavities, solution channels, or weathered seams below. As a result, the consumption of grout under such conditions was very low. Fig. 63 shows excavation to the key bed of shale and bentonite in the spillway

area. The smooth rock floor is clearly visible. Solution channels along the upstream face, at the left, have been walled up with concrete, the forms being still in place.

A general consolidation grouting program was developed with 30-ft wagon-drill holes on 10-ft centers. However, the rock was found to be sound and required so little grout that the hole spacing was changed to 12 ft and later to 20 ft. The total cement used for spillway and power-house consolidation grouting was 3,050 cu ft in 672 holes. Uplift gages were used when it was considered necessary.

For the cutoff grouting, the holes were drilled 4 ft downstream from the upstream face in two sets with both in the same vertical plane. The first

Fig. 64.—Power-House Excavation, Facing Upstream; Deep Solution Channel Opened Up Along Fault

set was drilled 40 ft at an angle of 25° in one direction. These were washed and grouted. The second set was drilled as a check but inclined in the opposite direction. These holes required relatively little grout. Many of the consolidation and cutoff holes scarcely took enough grout to fill them. The total quantity of cement used in the cutoff was 530 bags in 370 holes.

A large part of the power-house foundation was complicated by the occurrence of a rather flat fault, the outcrop in the final foundation extending almost diagonally across the downstream corner of the service bay at the south end to the upstream end of Unit No. 3. This fault dipped in an upstream direction. The service bay was excavated 30 ft deeper than would ordinarily have been required, due to the cavernous condition of the rock. This held true for the intake structure, but to a lesser extent. Fig. 64 shows the cavernous nature

of the rock under the intake. After the final bottom for excavation had been reached, a few mud seams were found in and above the fault for a distance of 25 ft from the outcrop. A part of this was excavated and the remainder was treated by repeating the washing and grouting several times.

In addition to the cutoff grouting with the wagon drills, deep holes, spaced 12 ft on centers, were drilled along the same cutoff line with a diamond drill, after the concrete was placed. The object of these holes was to penetrate the two thick layers of bentonite. All holes penetrated the upper layer, and the alternate holes were drilled through the lower bed. These holes were drilled alternately, the deep ones being treated first. No washing was done on these holes, since there were no mud seams. Higher pressures were used, to 125 lb per sq in. in the lower portions of these holes. Packers were set at the level of the bottom of the original cutoff grout curtain for the high-pressure grouting. Later, the upper section was grouted at about 60 to 75 lb pressure. The total quantity of grout used in the high-pressure holes in the spillway and power house was 1,100 cu ft of cement in 114 holes, which totaled 11,500 lin ft. In the entire power house and spillway, including cutoff, high-pressure, and consolidation grouting, 12,207 cu ft of cement were injected in 1,231 holes.

Temporary Construction Grouting.—The preliminary grouting on lines parallel to the cutoff trench under the south earth dam, for convenience in construction, has been mentioned previously. This grouting also had some beneficial effect on the permanent structure.

In addition, considerable cofferdam grouting was done. In the first-stage cofferdam surrounding the lock, there was no preliminary grouting before the cofferdam was pumped. As excavation progressed, particularly in the upstream end, leakage increased. A few preliminary exploration holes had been drilled previously to indicate the conditions on which the sheet-steel cofferdam was designed. According to the original interpretations, alternate layers of rock and filled cavities were found below the river bed of gravels. However, it developed that this section was a confused mass of clay, gravel, and large boulders or slabs of rock. These rock slabs were often quite large and offered an excellent roof for the percolation channels under them.

Grouting was done during the time the cofferdam area was unwatered, and after leakage had become serious. Holes were drilled inside of the cofferdam, extending from the upstream land connection to a point about one third of the way along the river arm. Most of the drilling was done with 5.5-in. shot drills. A large hole was considered advisable in order to force coarse materials through where necessary. Holes were tested with dye to determine what connections existed. Various types of grout were used. Sand, bentonite, and cement mixtures were used in many of the holes. To this mix, sawdust and shavings were added and even strips of burlap and small cloth sacks of bentonite. Asphalt and pitch were used also. From some of the holes, this material leaked into the springs in the excavated areas. Accordingly, a wide berm was built outside of the cofferdam cells; and clay, cinders, and hay were sluiced in, or placed over, areas in the river bed and nearby creek channel, where sacks, dragged along the bottom, indicated that leakage might have

started. Eventually, the leakage was reduced and the construction of the lock completed, but not without considerable trouble.

The total quantities of each type of grout used in grouting the first-stage cofferdam were as follows:

Item	Material	Cubic feet
1	Straight cement	1,480
2	Sand-bentonite-cement	19,900
3	Sawdust, etc., with Items 1 and 2	3,730
4	Asphalt and pitch	36,350

The piers supporting the upstream extension of the river wall of the lock were designed to be built directly on rock. Conditions similar to the foregoing, under the adjacent section of the cofferdam, prevented open excavation; and the pier footings were redesigned, using reinforced concrete columns. This difficult operation also required much grouting to check inflows of water as holes were driven for the columns. These holes varied in depth from 40 to 80 ft.[17]

As a result of the experience in the first-stage cofferdam, a thorough geological study was made of the conditions under each of the two following stages prior to their installation. Additional holes had been drilled and the geologists were able to make an accurate interpretation of the foundation.

The decision was made to grout such areas as might develop leaks in advance of pumping out the cofferdam. The studies indicated that in both stages the upstream arms would require this treatment. Cofferdam No. 2, which surrounded most of the spillway, was grouted along the upstream and river arms. Geological studies indicated that grouting of the downstream arm would not be necessary.

In order to expedite the grouting and not delay pumping out, this work was started as soon as possible. In each cell, two 6-in. casings were set and braced to the steel piling prior to filling the cells with gravel. In each diaphragm connecting cell, one casing was installed, making the average spacing about 17 ft, center to center. This line of holes was located near the downstream side of the cells. Holes were drilled as fast as the cells were filled, and they were grouted as rapidly as possible. Alternate holes were drilled and grouted first to keep each operation from interfering with the other. During grouting, a close watch had to be maintained for direct leaks in the river. Where these leaks were found, grouting would be slowed up to allow the leak to plug. Frequently, calcium chloride was added to speed the setting; sawdust was also added. A limit of 500 sacks of cement per hole was established to avoid unnecessary waste.

The cofferdam was completed and pumped out before any regrouting could be done. Additional drilling was done inside of the cofferdam and adjacent to the cells, which avoided the additional expense of drilling from the tops of the cells, to check up on sections where the first grouting indicated that further treatment was necessary. This explains the reason for locating the casings in the cofferdam cells on the downstream side or inside face. In this

[17] *Proceedings*, Am. Soc. C. E., June, 1939, p. 953.

manner, both lines of grout holes were in approximately the same zone. A gravel berm around the inside face of the cofferdam added stability to the cells and gave a thicker blanket to grout against. No trouble with leakage was had with this cofferdam. Most of the water pumped out during the construction period was due to washing and concrete curing.

With accurate advance information on the third-stage cofferdam, which surrounded the remainder of the spillway and the power house, grouting was planned and executed carefully. Here again only the upstream arm was grouted. Geological studies again indicated that there would be no trouble from the downstream arm in this area.

The deep cavernous and faulted area that ran diagonally through the power house was under the upstream arm of the cofferdam and was thoroughly grouted. Practically all of the grouting for this stage was completed before unwatering.

Leakage in this cofferdam was very small. A flood of 185,000 cu ft per ft put a head of about 30 ft on these cells, and the only additional leakage was attributed to cell drainage. The inside berm along the cells had not been completed at that time.

Grouting of the cofferdams was considered well worth while. The actual pumpage was slight, and there was no interference with construction work due to an inadequate cofferdam. In addition to the grouting of the cells in the third-stage cofferdam, a large quantity of stabilized clay grouting was done from the land-tie cell for a distance of about 200 ft back from the river bank and connecting with a previously constructed section of the core trench, which has been mentioned previously. The following list gives the total quantities of material used in grouting the third-stage cofferdam:

Material	Cubic feet
Cement	30,826
Bentonite	6,355
Sand	37,796
Sawdust	7,393
Clay (in stabilized clay grout)	23,790
Total	106,160

Drilling Equipment.—Diamond drills were used for exploration work. The preliminary work was let by contract. After construction forces were on the job, two diamond drills were purchased for more detailed exploration and deep grouting. The best drill for the conditions at Chickamauga (where it was desirable to recover as much core as possible of the soft beds of bentonite and shale) was found to be the hydraulic-feed type, equipped with a double-core barrel. Bortz stones were used. The earlier drilling had been done with screw-feed drills and practically none of the soft beds was recovered. They had generally been logged as lost core or filled cavities.

Calyx shot drills were used where larger holes were wanted for grouting, particularly for asphalt and cofferdam sealing. In the latter case, sawdust and shavings might be necessary at any time for plugging open leaks, and the hole was necessary. These drills also were used, without shot, to install the

casing through overburden in advance of drilling into the rock with a diamond drill. The diamond drills were too light to install the casing as fast as could be done with the shot drills.

Large calyx shot drills were used for sinking shafts to deep cavities, for inspection of the rock, and for constructing the columns to support the piers in the upstream river wall of the lock. Sizes varied from 30 in. to 49 in. in diameter, with depths as great as 80 ft. The largest drill had a rated capacity of a 72-in. diameter, but was never used for holes larger than 49 in.

Wagon drills were used for most of the grouting. The standard maximum depth drilled was 40 ft. When the drill had to be placed on a platform several feet above the rock surface, due to crevices or narrow excavation requirements, 10-ft extensions were added to the steel.

Grouting Equipment.—Three grouting units were built and assembled on the job. They were similar to those described previously[18] by V. L. Minear, M. Am. Soc. C. E. Two duplex, double-acting, air-driven, reciprocating pumps were used on these units. The entire assembly was mounted on skids for portability. The pumps were arranged so that either one could be used. The piping permitted a quick change-over from one pump to another without disconnecting any lines. This was to assure continuous operation in case one pump needed repairs or became plugged. When moving the unit, it was necessary only to disconnect and connect the air, water, and grout lines and the plant was ready to run. Two grout lines were used, one for the grout discharge from the pump and the other as a return line from the grout header at the hole, discharging into the sump tank. If the hole took grout rapidly from the start, the return line would not be connected. Where grout was taken slowly, the return line allowed a sufficiently rapid flow through the grout lines to prevent settling and plugging. By a proper arrangement of valves, the operator could control the rate of feed and pressure on the grout going into the hole. Telephones were used for communication between the operator at the grout plant and the man at the hole, since the plant was generally situated to cover a radius of 500 ft, and at the same time be convenient to a roadway for the cement trucks. The aforementioned plant and equipment operated successfully and economically.

With sand-cement-bentonite and stabilized clay mixes, three grouting units of an older type were used. Each unit consisted of a two-compartment, open-top, horizontal mixer and a duplex, double-action grout pump. The mixers had a single shaft extending through both compartments. The bottom of the mixer was rounded so that the blades on the shaft mixed the bottom of each batch thoroughly. The shaft was turned by an air motor. The pump section was connected to the bottom of both compartments through a valve that permitted mixing in one compartment while pumping from the other. The water was measured by the depth in the mixer. There was no provision for screening lumps out of the grout, and with only a single pump, delays due to plugging and valve wear were frequent. This arrangement, however, was satisfactory for grouting with a slow-setting mix where delays would not mean the loss of a hole or incomplete grouting.

[18] *Civil Engineering*, November, 1936, p. 751.

Method of Operation.—All diamond-drill holes were logged carefully by an inspector. The notes and the cores were studied by a geologist. Wagon-drill holes were watched, during drilling, for evidence of mud seams. A feeler was used in searching for small seams and to check drilling data.

Before grouting, the holes were washed in groups. Notes were made on the characteristics of each hole so that when grouting was started the inspector and engineer in charge had all the available information on the underground conditions upon which to base the method of starting and conducting operations. Accurate records were kept during the grouting of each hole so that an intelligent interpretation could be made of the results, and to determine the sufficiency, or the next method of attack.

Rigid instructions for grouting were not given; instead, flexible rules for guidance were used, since all holes do not act alike. With information available as to how a hole or group of holes acted during washing operations, grouting was started with a mix as thick as it was considered would be taken freely at a pressure of about 75% of the limit allowed. From this point on, the inspector would vary the water-cement ratio by thinning it if the pressure went up and thickening it if it dropped. Under the concrete structures, uplift had to be watched and considered in determining the mix and pressure. Grout mixes varied in consistency from 5.0 to 0.6 water-cement ratio, measured by volume.

Conclusions

The foundation treatment at Chickamauga Dam falls into three divisions: (1) That under the earth-dam sections; (2) that under the concrete structures; and (3) that of construction, or temporary grouting. Realizing that nothing but the most thorough job would do, it became a question of devising the most economical method.

Under the earth-dam sections, it was necessary to balance the cost of excavation, plus the earth backfill, against the probable cost of grouting, and dig to the point where the sum of the two was the lowest. There were many locations where a grout crew could have spent money at the rate of $2,000 to $3,000 per day on an individual hole had this not been anticipated. As it was, a much smaller total sum was spent by excavating to the point where grout consumption was relatively small. The method of excavation, whether by open cut or by shafts, depended on the depth. The foundation for the earth dams was excavated, not with the idea of placing a concrete structure on it, but to secure a low-cost cutoff. The attempt was made to keep the grout in the line of the cutoff curtain and not over a wide zone. The sequence of drilling, washing, and grouting was decided with this end in view. If the same number of holes had been drilled and grouted over a wide zone, the cutoff line would not have been as tight as the method followed. It could be compared to building a picket fence. In the case of the wide zone, four fences could be established with the pickets wide apart; whereas, if all the pickets were built into a single fence, a tighter barrier would be established.

The conditions under the concrete structures, lock, spillway, and power house were different. A firmer rock was required. The occurrence of bentonite beds, with sound unweathered rock below, together with the depth

required for various portions of the structure, determined the limits of excavation under all of the concrete structures, with the exception of the upstream end of the lock and the south half of the power house.

In the case of the lock guide walls, where the hydraulic head would always be balanced, the requirements for the foundation were not as exacting as other parts of the structure. In the consolidation grouting that was applied to the entire foundation under concrete structures, more grout was used under the lock guide walls than elsewhere. This reduced the excavation and speeded the progress of the job as a whole. It was a question of balancing the cost of excavation (plus the concrete to fill it up) against the grouting costs, and still meet the requirements for soundness.

The necessity for temporary construction grouting was a result of experience gained in the first cofferdam, around the lock area, and in the cutoff trench excavation on the south side. The requirement was for a low-cost grout that was needed for temporary purposes only, since a large volume was anticipated. Many different materials were considered but the high costs ruled most of them out. The sand-cement-bentonite grout cost less than half that of straight cement. This combination mixture proved satisfactory. The stabilized cement-clay grout used for backfilling cavities under the earth dams is far more permanent than any material already existing in them, and the cost was one third less than the sand-cement-bentonite mix.

In conclusion, it is recommended that, for any dam site situated with similar foundation characteristics, an exploratory drilling program be planned with closely spaced holes. At Chickamauga Dam, the preliminary exploratory information was of little value until holes were drilled on 25-ft centers. Careful drilling must be done in order to recover as complete a core as possible. Accurate logging of holes is necessary.

ACKNOWLEDGMENT

H. E. Murray, associate engineer, supervised the field grouting operations under the writer's direction and assisted the writer in preparing this paper.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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DISCUSSIONS

BEACH EROSION STUDIES

Discussion

BY EARL I. BROWN, M. AM. SOC. C. E.

EARL I. BROWN,[11] M. AM. SOC. C. E. (by letter).[11a]—In general, the publication of this paper has resulted in beneficial discussion.

Mr. Lipp discusses the paper from the standpoint of his Florida experience, and gives the reasons for his practices on that shore. He stresses the difficulties encountered at inlets, those arising from heavy surf action, and the other local conditions to which his discussion calls attention. He finds, in general, that his experience corroborates and confirms the views expressed in the paper.

Mr. Lipp's statement that, in any beach erosion study, "It is first necessary to know why certain problems exist, whether they are Man made or made by Nature," is true. It is also true that the engineer must usually accept conditions as he finds them and solve the problems presented thereby. In his solutions of beach protection problems at shore communities, the engineer can do much toward increasing the recreational uses of the beaches and reducing the hazard and destruction in storms by establishing the essential information on which the local government should base its zoning ordinances.

In almost all cases where inlets have been cut through sand barrier beaches and protected by jetties, there has been an ensuing accretion on the one side of the inlet and erosion on the other, as observed by Mr. Lipp. Pumping from the "full" side to the eroded side has been advocated by some individuals for many years, but the only location known to the writer at which it has been tried is that cited by Mr. Lipp. The possibilities involved merit additional study and experimentation.

The starving of beaches to the leeward of a well-built system of groins is familiar to all students of the subject of beach erosion and is another argument in favor of comprehensive planning for the development and protection of entire beach areas. To take care of the original filling of groin bays, it is

NOTE.—This paper by Earl I. Brown, M. Am. Soc. C. E., was presented at the meeting of the Waterways Division at Jacksonville, Fla., on April 21, 1938, and published in January, 1939, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: April, 1939, by Morris N. Lipp, M. Am. Soc. C. E.; May, 1939, by Messrs. George A. Soper, and James J. O'Rourke; June, 1939, by Messrs. Elliott J. Dent, and Ralph F. Rhodes; and February, 1940, by Charles T. Leeds, M. Am. Soc. C. E.

[11] Col., Corps of Engrs., U. S. Army (*Retired*), Wilmington, N. C.

[11a] Received by the Secretary January 17, 1940.

advisable, of course, to start the construction of a system of groins with the most leeward. Unless there is known to be a large littoral drift, the necessity of maintaining the beach by artificial filling between groins must always be considered when developing a plan of protection.

Mr. Soper expresses opinions probably held by many practicing engineers concerning the elaborateness of the investigations recommended in connection with beach erosion studies. It is probably true that many problems in connection with shore protection have been solved successfully "at the least expenditure of time and money." On the other hand, the practice of designing and constructing beach protective works on little or no real information is dangerous and has been the cause of the wasting of millions of dollars. It is the writer's opinion that much of this loss might have been saved if the investigations had been more complete. The recommended investigations prior to construction (particularly as to geology, shore-line and offshore changes, wave action and storm effect, tidal and other current conditions, character and extent of littoral drift) are as necessary as foundation investigations for buildings. If direct data are not obtained, practicing engineers are forced to base decisions on criteria taken from existing construction which has proved successful. The weaknesses of this condition are well known to the engineering world. It is to be understood that the scope of the investigations to be made in each particular study will be based on local conditions, available information, and the problem involved. Each of the different investigations should be made only when it is known that the results will have an important bearing on the problem.

Mr. Soper, very properly, emphasizes the desirability of inquiries into the history of destructive storms and their effects on structures. He also suggests the value of personal history as to local conditions. In these views the writer concurs fully. He states that the paper does not discuss, adequately, the movement of sand under the influence of the wind. This omission is acknowledged, and is to be attributed to the fact that few observations are available on this subject. The most satisfactory known observations on the relation between velocity of wind and volume of sand moved were made by the U. S. Engineer Office at Portland, Ore., in connection with a study of the movement of sand at the mouth of the Columbia River. A sketch of sand traps developed in connection therewith is given in Fig. 20 (prepared from a description of the trap used by Morrough P. O'Brien, M. Am. Soc. C. E., and B. D. Rindlaub, Lieutenant, Corps of Engineers, in tests at Clatsop Beach, Ore.[12]). When it is recalled that the sand dunes along the coast are formed from wind-blown sand, its importance in the study of coastal phenomena may be appreciated. Although the removal of sand from the beach by wind undoubtedly affects the rate of erosion, statistical data are meager. Mr. Soper also queries whether any method of removing the water behind a bulkhead can be suggested. The writer believes that every effort should be made to prevent water from seeping behind a sea-wall or bulkhead, but, when once there, no satisfactory way to remove it is known except to provide a suitable opening for drainage in the wall, so constructed as to prevent the escape of back-fill material. The writer deems that a bulkhead that is not completely back-filled is not a complete

structure, and that if the economics of the problem do not justify the cost of a back-fill, some other solution of the problem should be sought.

Mr. O'Rourke has made a considerable contribution to the art of shore protection in designing his submerged bulkhead. This structure appears to be

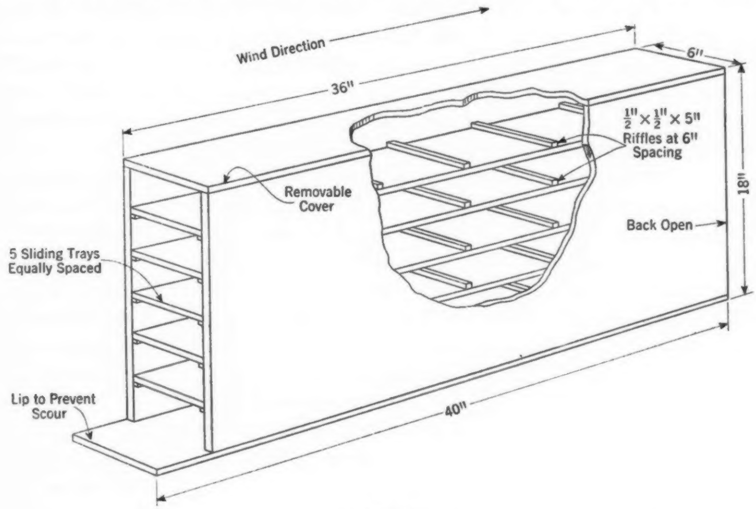

FIG. 20

well suited for conditions on the Great Lakes where it was constructed. There may be some question as to its effectiveness on an ocean beach where tidal variations may reduce its beneficial effects at the higher stages of the tide. An attempt to accomplish a similar result on the ocean beach has been devised experimentally by Mr. Richard Arpen, in the form of a triangular concrete block termed a "submerged breakwater." It appears to have had some beneficial effect in trials.

Mr. O'Rourke's description of the partly submerged bulkhead is of great interest. In several instances known to the writer, the construction of offshore breakwaters has been followed by a building out of the beach behind it. As a general rule, the high cost of offshore structures precludes their use solely for the purpose of beach protection. An experimental study of such structures has been made by the Beach Erosion Board in its experimental laboratory at Washington. The preliminary results appear to justify further study, both in the laboratory and the field.

Colonel Dent and Major Leeds question the rules used by the Beach Erosion Board in determining the ratio of length of groin to interval between groins. These rules are entirely empirical and cannot withstand logical analysis. However, observation of installed groin systems indicates that the rules may be relied upon, generally, to give a reasonable layout for a groin system. These writers also raise the point as to whether the stated length of groin is the total length or the length protruding after a sand fill has accumulated. It is cus-

[12] An unpublished report to the District Engineer, Portland, Ore.

tomary to consider the length of structure as built; a groin system functioning with 100% efficiency would be practically buried by the accumulated sand.

Colonel Dent states that the best spacing may depend on a balance of the cost of a more generous supply of nourishment (presumably by artificial means), more frequent points of supply, stronger bulkheads, more groins, and the general utility of the improvement finally adopted—that is, on the economics of the problem. There are many cases where economical considerations require the adoption of a project for erosion control less expensive than the ideal solution. Such a case has arisen in the writer's experience in designing a beach-erosion control project for the Town of Wrightsville Beach, N. C.

The frontage to be controlled was about 12 000 ft long, and the cost had to be kept within the sum of $250 000. The ocean was rapidly encroaching on, and endangering, existing structures, and it was desired to provide a safe margin of protection as rapidly as possible. The design made by the Beach Erosion Board for the locality comprised a groin and bulkhead system estimated to cost about $750 000, but such a sum was entirely beyond the capacity of the locality to finance. Its legal bonding limit confined its efforts to the aforementioned sum. The design adopted to meet these limitations consisted of an extension of the berm seaward for distances of from 100 to 200 ft by sand artificially supplied by dredging from the lagoon in the rear of the island on which the town is located, and the fixation of the artificially created beach by a system of sixteen groins. The latter are of creosoted timber constructed in general accordance with the Beach Erosion Board's typical design, with some modification, particularly at the outer end, to prevent erosion by eddies around the ends. The inner part of the groins is not horizontal but slopes from Elevation 9.5 ft (M.L.W.) at the landward end to Elevation 8.5 ft at a point 90 ft seaward; thence it slopes for 140 ft down to Elevation 2.0 ft; and then extends seaward horizontally for 95 ft, making the total length of groin 325 ft. It was possible to provide only sixteen groins to cover the distance of 12 000 ft; hence the spacing was about 800 ft. The volume of sand supplied to extend the berm seaward as desired, and to fill the intervals between groins substantially to the tops of their inner and middle sections, was 700 000 cu yd. The groins were practically buried within the fill since their function was to stabilize the berm and not to create one by accumulating drift. This project is functioning satisfactorily.

Mr. Rhodes discusses the paper in the light of his experience on the Georgia coast where he finds beach problems very intimately involved with those of currents at inlets, and their subsidiary effects. He finds that, in general, the experience and practices described in the paper correspond with his experience. He points out that at, and in, inlets the rules for spacing groins should be modified to require quite close spacing, as the problem there becomes one of bank protection rather than one of shore protection.

Major Leeds also finds it necessary to make an extended research into the past history of a beach under study in formulating a design for protective works, and he stresses the necessity for the use of good judgment in interpreting the available data. He stresses in particular the desirability of the fullest and

most complete data possible as to winds and waves. He considers wave data more important than wind data.

Major Leeds advocates the study of the residual movement of some material placed on the beach, of approximately the same specific gravity as the beach sand or gravel, but readily distinguishable therefrom, and a periodic observation of its aggregate movement. The Beach Erosion Board has attempted to make such studies,[13] using a red sand in lots of 16 to 20 cu yd each, placed in a trench extending from the crest of the berm to mean sea level. It found that this material became so diluted by mixture with other materials that its travel could not be followed along the beach for any considerable distance, and that in a short time it could not be identified at all. Further experiments with larger volumes of foreign materials might yield further information.

Major Leeds calls attention to a matter of grave concern to beaches on deeply indented and rocky shores, such as those of California. Many of these beaches are fed by detritus brought down to the sea by mountain torrents. The improvement of the valleys of these torrents in the way of flood-control and erosion-control projects is likely to reduce or eliminate such supplies, resulting in great damage to the beaches.

Major Leeds makes many valuable suggestions on groin design, some of which do not agree in detail with the recommended design. These comments are appreciated, although it was conceded in the paper that many variations in design of groins are permissible under certain conditions.

Regardless of the methods adopted, the cost of beach protection is high. Engineers studying beach problems should take full cognizance of the necessary relationship between the cost of protection and the value of the property to be protected. The general public readily endorses the construction of a shore boulevard or boardwalk at costs of $25, $50, $100, or possibly more per running foot. The necessity of proper expenditures for the protection of the beach itself should be apparent but it is often disregarded. There are more than sufficient examples to warrant the advocated principle of the Beach Erosion Board that upon decision to build, it should be axiomatic to build carefully and strongly. This does not prevent the possibility of savings in construction costs based on intimate and detailed knowledge of local conditions probably beyond the scope of a study by the Board.

As stated in the original paper, the science or art of shore protection is comparatively new. Sound judgment based on careful observation and critical analysis of existing beaches and structures is, and will remain for many years to come, the principal tool of the beach protection engineer. Quantitative methods of predicting any aspect of shore-line behavior are almost nonexistent and the subject is of such nature that it will not soon be presented in handbook form.

Shore-line behavior, however, is uniquely related to the hydraulic phenomena at work and the difficulty lies not in mysterious phenomena or insuperable difficulties in observation and interpretation but rather in the extreme variability and uncertainty regarding the sequence and magnitude of the winds, tides, currents, and sand supply. Hydraulic models of typical or specific

[13] Interim Report, Beach Erosion Board, April 15, 1933, paragraphs 3/1 to 3/6, inclusive.

situations, if built to sufficiently large scale, would undoubtedly yield reliable data concerning major phenomena but one is always uncertain about the manner in which the controlled variables are to be introduced.

The large number of variables involved in every study of the forces acting upon the shore line emphasizes the necessity for comprehensive planning after thorough consideration of all detailed information that can be secured, in order that there will be no possibility of overlooking any clue to the best possible solution.

In conclusion the writer wishes to express his appreciation to those who have discussed the paper, and his indebtedness to the Beach Erosion Board for valuable assistance.

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DISCUSSIONS

FUNCTIONAL DESIGN OF FLOOD CONTROL RESERVOIRS

Discussion

By Messrs. J. C. Stevens, Edward J. Bednarski, Ronald A. Kampmeier, and Edgar E. Foster

J. C. Stevens,[9] M. Am. Soc. C. E. (by letter).[9a]—By substituting a uniform inflow for the variable inflow of the actual flood, the authors of this paper propose to simplify the problem of routing floods through reservoirs. This uniform inflow is selected by trial to have such a duration T that it will leave the same volume stored in the reservoir, and produce the same maximum rate of outflow as the actual flood event. It cannot determine any rate of outflow other than the maximum; yet other values of the outflow during the flood are frequently of considerable importance. Neither can it determine the time of occurrence of the actual outflow, which is generally of paramount importance.

The question arises as to whether the simplification resulting from this method of approach is not more apparent than real, and whether one might not solve the problem for the actual flood event in less time than that taken to find the "equivalent uniform flood," particularly if it is important to know the actual pattern of outflow.

For example, consider the basic relationship given in Equation (4). Since both the outflow o and the storage capacity s are single-valued functions of the depth above the outlet h, the latter may be eliminated from the equations and, therefore,[10] $s = f(o)$ from which

$$\frac{ds}{do} = f'(o) \qquad (27)$$

but $f'(o)$ represents the slope of the curve $f(o)$ at any given point. If a curve is plotted, or if a table of storage increments corresponding to outflow increments at various values of outflow is compiled, it is possible to obtain the function

Note.—This paper by C. J. Posey, Jun. Am. Soc. C. E., and Fu-Te I, Esq., was published in October, 1939, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: December, 1939, by Edward Soucek, Jun. Am. Soc. C. E.

[9] Cons. Hydr. Engr. (Stevens & Koon), Portland, Ore.

[9a] Received by the Secretary January 24, 1940.

[10] "Computing Reservoir Outflow and Height from Inflow and Capacity," by J. C. Stevens, *Engineering News-Record*, December 22, 1921, p. 1031, Equations 2 and 3.

$f'(o)$ and, once for all, have the essential data to route any flood through that reservoir.

Let t = any finite time interval; i = average inflow during that interval; o_1 = outflow at the beginning of that interval; o_2 = outflow at the end of that interval; and θ = slope of the outflow-storage curve as previously explained for the average value of outflow during that interval; that is, $\frac{ds}{do} = f'(o) = \theta$.

It is well to fix the significance of θ well in mind. It merely means that for each cfs-increment (cfs = cubic feet per second) in outflow the storage increment is θ sfd (second-foot-day; increment may be either + or −).

Substituting Equation (27) in Equation (4)

$$i\,dt = o\,dt + f'(o)\,do \qquad (28)$$

Equation (28) may be translated into the finite form

$$o_2 - o_1 + \frac{(o_2 + o_1)t}{2\theta} = i\frac{t}{\theta} \qquad (29)$$

Solving for the outflow at the end of the time interval t:

$$o_2 = \frac{i}{\frac{\theta}{t} + \frac{1}{2}} + o\,\frac{\frac{\theta}{t} - \frac{1}{2}}{\frac{\theta}{t} + \frac{1}{2}} \qquad (30)$$

Starting with an initial outflow o_1, the outflow o_2 is found directly, which then becomes the o_1 for the next interval.

The ease and rapidity with which this formula is applied in a step-by-step process, and the accuracy of the outflow pattern obtained thereby, rather obviate the necessity of trying to find an equivalent uniform inflow. This point so deserves emphasis that the writer believes an example of its application is warranted.

TABLE 4.—VALUES OF θ FOR USE IN ROUTING A FLOOD

Description	LAKE LEVELS (FROM ASSUMED DATUM):										
	28	29	30	31	32	33	34	35	36	37	38
Outflow,* o	15.5	17.7	20.1	22.7	25.6	29.1	33.0	37.0	41.2	46.0	51.0
Difference, Δo		2.2	2.4	2.6	2.9	3.5	3.9	4.0	4.2	4.8	5.0
Storage,† S	185	210	235	262	289	316	343	370	398	426	454
Difference, ΔS		25	25	27	27	27	27	27	28	28	28
Ratio, $\frac{\Delta S}{\Delta o} = \theta$		11.3	10.4	10.4	9.3	7.7	6.9	6.7	6.7	5.8	5.6

* In thousands of cubic feet per second. † In thousands of second-foot-days.

To illustrate, the flood of November-December, 1917, is routed through Coeur d'Alene Lake, Idaho. This lake forms the head of the Spokane River. At the outlet, a rating curve has been developed. The control is the 8 miles of river between the lake and Post Falls, Idaho. The storage capacity of the lake

has been determined by surveys. The principal tributaries are the Coeur d'Alene and St. Joe rivers on which inflow was measured and a small percentage added for other tributaries. Table 4 gives the data necessary to route the flood. These data, with any given inflow, are all that is required to route any flood through this lake.

Table 5 shows the computations for routing the flood of November-December, 1917. The average inflow for each time interval in Table 5 was read

TABLE 5.—COMPUTATIONS FOR ROUTING THE FLOOD OF NOVEMBER-DECEMBER, 1917, THROUGH COEUR D'ALENE LAKE, IDAHO

Date (1917)	HOUR*		Period, t, in days	In-flow,† i	$\frac{\Delta s}{\Delta o} = \theta$	$\frac{\theta}{t} + \frac{1}{2}$	$\frac{\theta}{t} - \frac{1}{2}$	$\frac{i}{\frac{\theta}{t} + \frac{1}{2}}$ (cfs)†	$\frac{o_1 \text{ times Column (7)}}{\text{Column (6)}}$ (cfs)†	Out-flow,† o_2	Lake level, in feet
	From	To									
	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)
(November)											
25	0	24	1	....	....	....	....	....	....	21.2	30.4
26	0	24	1	13.0	10.0	10.5	9.5	1.2	19.2	20.4	30.1
27	0	24	1	20.0	10.0	10.5	9.5	1.9	18.4	20.3	30.1
28	0	12	0.5	31.0	10.0	20.5	19.5	1.5	19.3	20.8	....
28	12	24	0.5	41.0	10.0	20.5	19.5	2.0	19.8	21.8	30.7
29	0	6	0.25	47.0	9.5	38.5	37.5	1.2	21.2	22.4	....
29	6	12	0.25	51.0	9.5	38.5	37.5	1.3	21.8	23.1	31.2
29	12	18	0.25	56.0	9.5	38.5	37.5	1.4	22.5	23.9	....
29	18	24	0.25	60.5	9.0	36.5	35.5	1.7	23.2	24.9	31.7
30	0	6	0.25	68.0	9.0	36.5	35.5	1.9	24.2	26.1	....
30	6	12	0.25	72.5	8.5	34.5	33.5	2.1	25.3	27.4	32.5
30	12	18	0.25	76.0	8.5	34.5	33.5	2.3	26.4	28.7	....
30	18	24	0.25	78.0	8.0	32.5	31.5	2.4	27.8	30.2	33.3
(December)											
1	0	6	0.25	79.0	8.0	32.5	31.5	2.4	29.2	31.6	....
1	6	12	0.25	78.7	8.0	32.5	31.5	2.4	30.6	33.0	34.0
1	12	18	0.25	78.0	7.5	30.5	29.5	2.6	31.9	34.5	....
1	18	24	0.25	75.0	7.0	28.5	27.5	2.6	34.3	36.9	35.0
2	0	12	0.5	68.0	6.5	13.5	12.5	5.0	34.2	39.2	....
2	12	24	0.5	62.0	6.0	12.5	11.5	5.0	36.0	41.0	35.9
3	0	12	0.5	57.5	6.0	12.5	11.5	4.6	37.7	42.3	36.2
3	12	24	0.5	53.0	6.0	12.5	11.5	4.1	38.8	42.9	36.3
4	0	12	0.5	48.0	5.5	11.5	10.5	4.2	39.2	43.4	36.4
4	12	24	0.5	41.5	5.5	11.5	10.5	3.6	39.6	43.2	36.4
5	0	24	1	35.0	5.5	6.0	5.0	5.9	36.0	41.9	36.1
6	0	24	1	30.0	6.0	6.5	5.5	4.6	35.4	40.0	35.7
7	0	24	1	28.0	6.0	6.5	5.5	4.3	33.4	37.4	35.1
8	0	24	1	26.5	6.5	7.0	6.0	3.8	32.0	35.0	34.5
9	0	24	1	24.0	7.0	7.5	6.5	3.2	30.4	33.6	34.2

* Midnight = 0. † In thousands of cubic feet per second.

from the inflow curve of Fig. 3. Values of θ were also read from Fig. 3, taken to the nearest 0.5 only. The outflow o_2 in Column (10), Table 5, is the sum of columns (8) and (9) and applies to the end of the periods indicated, the preceding value being the outflow o_1 at the beginning of the period as used in Column (9). Lake heights (Column (11)) are read from Fig. 3, using outflow as the argument.

The simplicity and elasticity of this method is apparent at once. Time intervals may be varied at will, using short periods when the inflow is varying rapidly and longer periods when the inflow is steady. One may start at any known lake height or inflow. The only auxiliary curve used is the θ-curve of Fig. 3, values being read to represent the slope for the average outflow during each period.

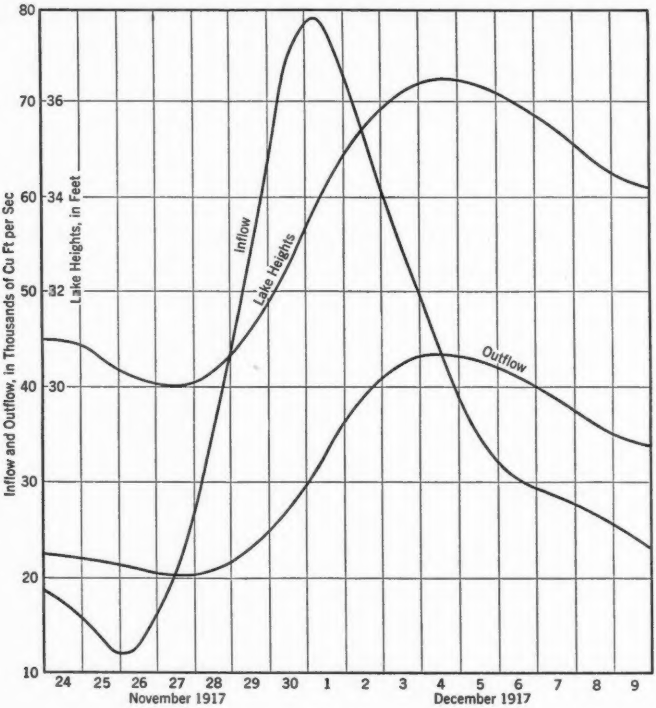

FIG. 3

The writer would like to emphasize the advantages of the second-foot (cfs) for flow and second-foot-day (sfd) for storage over other units. Inflows are usually tabulated in second-feet (cubic feet per second), which avoids a conversion; also, sfd = half the number of acre-feet, both being units of volume.

EDWARD J. BEDNARSKI,[11] ASSOC. M. AM. SOC. C. E. (by letter).[11a]—The functional design of flood control reservoir, like any other design, is: (1) A process of analyzing the conditions and requirements that the structure must satisfy; and (2) a creative part, or a synthesis, producing a design that will fulfil the requirements of the problem.

The problem of flood control requires a thorough analysis of meteorological, hydrological, topographical, and geological properties of a locality in need of flood control. An economic analysis should also be included regarding the magnitude and character of potential damage expected from the absence of flood control. This work should produce the following items for the use in the creative part of the hydraulic design:

(1) A flood hydrograph, to be considered as a controlling factor of the design;

[11] Structural Engr., Los Angeles, Calif.

[11a] Received by the Secretary February 5, 1940.

(2) A maximum rate of runoff within the watershed of the locality which could be tolerated under existing, or properly improved, conditions; and

(3) The topographic and economic data pertaining to every possible reservoir site in the form of elevation-capacity curves and the highest allowable water elevation.

The problem confronting the designer is the selection of a site for a reservoir which satisfies the foregoing stipulations and, if it does, the determination of the size and the type of the outflow facilities.

The authors present a method based on a mathematical treatment of a differential formula (Equation (4)). In order to do so they disposed of the equation

$$i = f(t) \qquad (31)$$

assuming that it can be substituted by

$$I = \frac{\int_0^T i\,dt}{T} \qquad (32)$$

an average rate of inflow. The outflow, however, is considered variable, starting from zero; then it reaches a maximum Q which is not supposed to be larger than I. Contrary to the notation of the paper the writer uses q and Q instead of o and O, respectively.

Unfortunately the writer has met with a number of cases in which $\frac{Q}{I}$ was greater than 1.

In the light of experience it appears quite futile to attempt a mathematical solution of the problem. A better approach seems to be a study of Equation (31) as it is represented by a hydrograph, in the form of Equation (32), or a mass curve that can be obtained from the hydrograph by numerical or graphical summation.

Quite contrary to the assumptions of the authors are the conditions of the problem to determine the least possible constant rate of outflow for a given amount of storage. Here q is constant but i is variable; and in Equation (5) $n = 0$ and $q = Q$, ($q = B\,h^n$).

This problem is solved easily, as follows: Consider the mass curve (*a*) shown in Fig. 4. Draw the same mass curve (upper part of it) downward from the first by a distance numerically equal to the maximum available storage (curve (*b*)). A straight line between these two curves tangent to each (see curve (*c*)) solves the problem. The tangent of the angle that this line makes with the horizontal axis, expressed in terms of acre-feet-per-hour yields the answer. The diagram is self-explanatory.

The foregoing solution is applicable when the outlet conduits are so located as to have sufficient head to pass the required discharge when storage in the reservoir is assumed to be equal to zero. However, if a certain initial storage is required to build up the necessary head, the second mass curve (*b*) should be drawn below the first by a distance equal to the difference in storage between

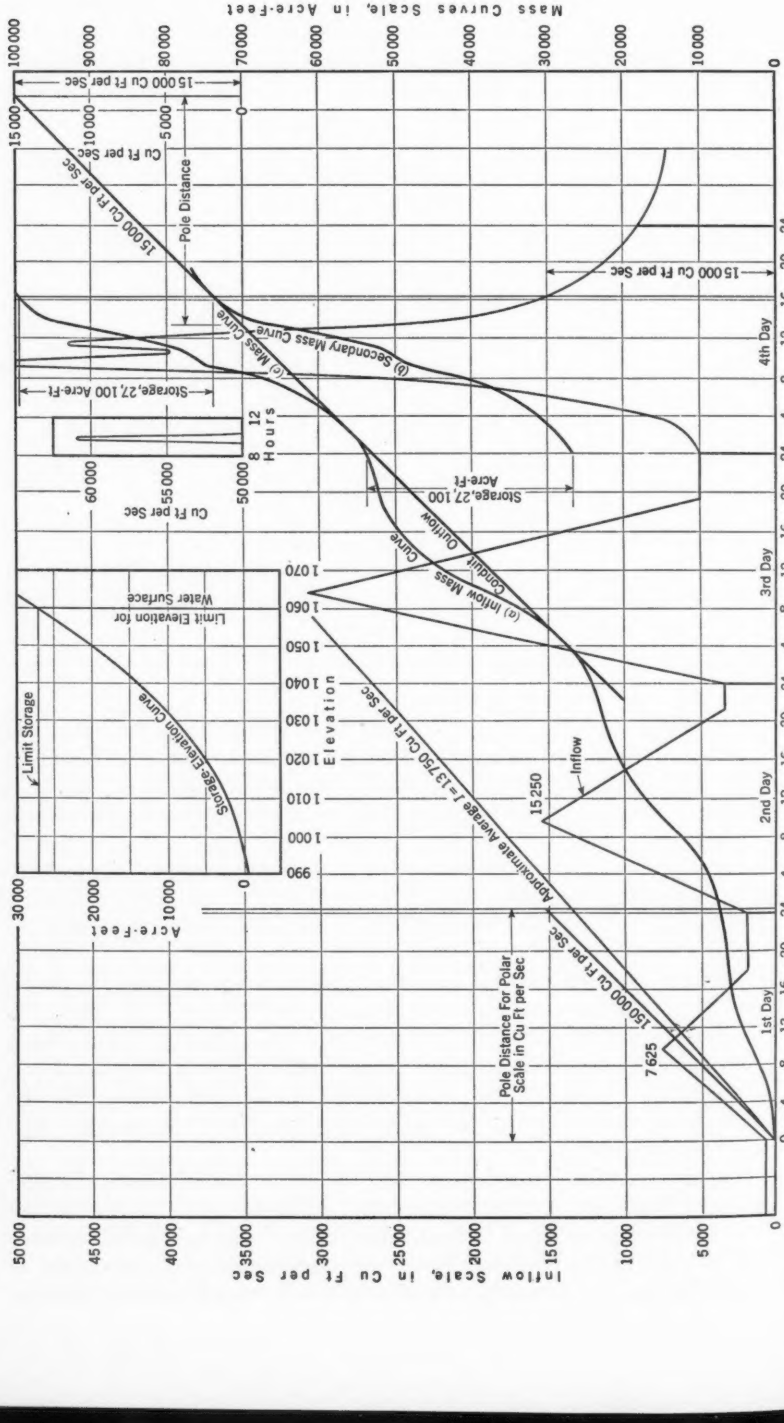

FIG. 4.—TO DETERMINE THE LEAST POSSIBLE CONSTANT RATE OF OUTFLOW FOR A GIVEN QUANTITY OF STORAGE

the required initial and the maximum available storage. A straight line (*c*) tangent to these two curves will again produce the solution of the problem.

The problem of spillway length and the elevation of the spillway crest cannot be solved so simply. The outflow is not constant any more and all that is known about the outflow mass curve is that it is tangent to the mass curve or the horizontal axis at the time t_0, and parallel with the tangent to the mass curve at t_e, the time when the outflow is expected to be equal to the inflow. Between these two points the curve has no point of inflection and the angle of the tangent to this curve at any point is smaller than a tangent to the inflow mass curve at the same time. With some practice, it is possible to predict whether there is any possibility of designing an ordinary spillway with a given highest permissible elevation of the water surface and the elevation of the spillway crest.

Using Equations (1) and (5) it is possible to determine the necessary length of the spillway crest embodied in the constant B, as follows:

$$B = \left(\frac{C}{S}\right)^p Q \qquad (33)$$

in which (to simplify typography) $p = \frac{n}{m}$. This may or may not be the solution.

Whether the spillway length thus determined is satisfactory could be found by means of mass and storage-elevation curves by arithmetical computation or graphically.[12] Evidently there is no general solution. Each case must be solved by trial and error, which is not unusual and many, if not most, engineering problems are solved in this manner.

The limits of this discussion do not permit a detailed description as to the use of mass and storage-elevation curves in order to save some of the drudgery connected with the hydraulic design, and the writer regrets to state that an ingenious idea of using an average value of the inflow rate (as suggested by the authors) does not relieve the designer from a mass of tedious and painstaking labor, unavoidable in the process of the solution by trial and error.

ROLAND A. KAMPMEIER,[13] JUN. AM. SOC. C. E. (by letter).[13a]—This interesting and worth-while paper extends the principle of Mr. Woodward's "five-sixths rule"[3] to a wide variety of possible reservoir conditions.

The "five-sixths rule" states that the ratio, r, of average outflow to maximum outflow, will be approximately five-sixths under certain conditions—namely, when the reservoir has an orifice-type outlet; when the exponent, m, in the depth-capacity relationship is approximately 2.5; and when the ratio, x, of maximum outflow to assumed constant inflow is not too large. From Table 2 and Equation (20) it may be determined that their cases cover a range

[12] See, for instance, "Hydraulik," by Ph. Forchheimer, Verlag und Druck von B. G. Teubner, Leipzig und Berlin; August 3, 1930, pp. 416, 417; or "Graphical Analysis of Spillway Capacity," by E. J. Bednarski, *Western Construction News*, February 25, 1932, p. 113 (see also erratum, *loc. cit.*, March 25, 1932, p. 168).

[13] Associate Prof., Hydr. Eng., Univ. of Tennessee, Knoxville, Tenn.

[13a] Received by the Secretary February 5, 1940.

[3] "Hydraulics of the Miami Flood Control Project," by Sherman M. Woodward, Technical Reports of the Miami Conservancy District, State of Ohio (1920), Part VII, Chapter XII, p. 286.

of values of r from 0.67 to 0.94 for the orifice-type outlet and from 0.41 to 0.85 for the weir-type outlet, the lowest values of r corresponding to the lowest values of m and of x. The effect of m is much more pronounced than the effect of x.

It may be noted that the use of weir-type outlets, to the exclusion of orifice-type outlets, would be unusual among reservoirs for flood control only (to the discussion of which the paper is limited), and that the paper indicates that the "five-sixths rule" is fairly satisfactory for orifice-type outlets when m is in the usual range of 2.0 to 3.5, and when x is less than about 0.6 (which would probably be the usual case in reservoirs designed exclusively for flood control).

Since the effect of the exponent m on the ratio r is so pronounced, it may be useful to emphasize that for a given reservoir the same value of m would not be likely to apply in comparing orifice-type and weir-type outlets. A weir-type outlet, or spillway, would normally be placed at a considerably higher level than orifice-type outlets, or sluices. If Equation (1) represented the depth-capacity relationship above the level of the sluices, the depth-capacity relationship above a higher spillway level could be represented approximately by a similar equation only if a smaller value of m were used. It would be interesting to know whether the values of m between 1.03 and 4.74, found by the authors for more than 250 reservoirs, were based on the depth-capacity relationships above sluice or spillway levels, as the case may be (as might be assumed from the applications in this paper), or above the bottom of the reservoirs, applying the equation as used by Mr. Sutherland.[7]

The authors mention (but do not discuss in detail) their method of selecting an equivalent uniform flood to replace the assumed design hydrograph, a method which differs from that used by Mr. Woodward,[6] who considered only that part of the hydrograph preceding the instant when the outflow equals the inflow and the maximum storage capacity has been reached. The authors' example (see Fig. 1) does not indicate whether or not a shorter equivalent flood of greater intensity would have been used if less storage capacity had been assumed available. The impression is left with the writer that the authors would use the same equivalent flood for all assumed conditions of outlet size and storage capacity. Suppose that the maximum outflow in the case indicated in Fig. 1 were 8,050 acre-ft per hr; using the equivalent flood shown would mean assuming an apparently unsatisfactory value of x of 1.1.

The writer questions whether the authors have used properly Mr. Woodward's term, the "protection ratio," which they state to be the same as their "outflow ratio," x, the ratio of the maximum outflow to the uniform inflow of the equivalent flood (see "Introduction: Outflow Ratio"). The writer understood the original use of the term "protection ratio" to mean the ratio of the maximum flood inflow to the maximum outflow,[14] whereas the term "outflow

[7] "Some Aspects of Water Conservation," by R. A. Sutherland, *Transactions*, Am. Soc. C. E., Vol. 96 (1932), p. 164.

[6] "Hydraulics of the Miami Conservancy District," by Sherman M. Woodward, Technical Reports of the Miami Conservancy District, Part VII, Chapter VII.

[14] *Loc. cit.*, Fig. 73, p. 197.

ratio" was originally used[15] with a different meaning than that assigned to it by the authors.

Appreciation of the data presented is obscured by the fact that the term x, in Table 2 and in Equation (3), does not reflect the actual reduction in maximum discharge. It might have been desirable to use values of the "protection ratio," as the writer understands its meaning, as headings of the columns in Table 2, instead of the values of the authors' "outflow ratio." The writer realizes that the use of the protection ratio would require assumptions as to the shape of the hydrograph. The isosceles-triangle form of hydrograph would seem to be a reasonable assumption somewhat more similar to the true hydrograph in most instances than the equivalent rectangular flood.

EDGAR E. FOSTER,[16] ASSOC. M. AM. SOC. C. E. (by letter).[16a]—The authors have undertaken to furnish a much-needed means of analyzing problems arising in regard to the functional design of reservoirs. There are three general problems to solve in functional design—namely,

(*a*) Design flood; that is, the magnitude and hydrograph of a flood upon which the operation of the reservoir is to be based;

(*b*) Outflow attainable; and

(*c*) Capacity needed to obtain the desired measure of control.

The reservoir design flood is built upon the quantity of rainfall in a suitable storm, by means of distribution factors; or it is derived from a selected flood (perhaps the maximum) experienced on the watershed. In any case, it is based upon conditions not touched upon by the authors' paper and hence need not be discussed further herein. The reduction of outflow (Problem (*b*)) and storage required (Problem (*c*)) are the questions with which the authors deal. These two factors are reciprocal in that when one is given the other may be obtained.

The problem of the reduction of outflow may be decided from the channel-carrying capacity below the reservoir, or some other critical point downstream. Under these circumstances there remains the problem of determining how much capacity is needed to reduce the design flood to the desired maximum outflow. On the first glance it seems that the authors had provided a method to yield the answer. Unfortunately, however, it is not possible to find the "equivalent rectangular" flood as readily as is indicated in the paper. The only sure method of determining the equivalent rectangular flood is that of routing the design flood through the reservoir; and, when that is done, the required capacity is known, and it becomes unnecessary to apply the method, since all elements are then known.

The writer made an attempt to determine the capacity of a reservoir under investigation by the authors' method. The pertinent data for the reservoir are as follows: Reservoir design flood, peak discharge, 32,400 cu ft per sec; volume, of flood, 311,000 acre-ft; total duration, of flood, 12 days; outlet discharge fixed at 10,000 cu ft per sec; total capacity of reservoir, 545,000 acre-ft; and m,

[15] "Hydraulics of the Miami Conservancy District," by Sherman M. Woodward, Technical Reports of the Miami Conservancy District, Part VII, Chapter VII, p. 200.

[16] Associate Engr., U. S. Engr. Office, Omaha, Nebr.

[16a] Received by the Secretary February 6, 1940.

derived from the capacity curve, 3.15. The problem is to fix the capacity to be set aside for flood control in order to reduce the design flood from 32,400 to 10,000 cu ft per sec.

On the basis of a 12-day period, the equivalent rectangular flood is 13,000 cu ft per sec, from which $x = 0.77$. Then from Table 2, by interpolation, $d = 0.316$. The capacity needed, therefore, should be $0.316 \times 311{,}000$ or 98,000 acre-ft. However, by routing this flood through the reservoir, it was found that approximately 150,000 acre-ft were needed. An attempt was made also to utilize the time (6.2 days) during which the design flood was above the maximum outlet discharge. This duration of the rectangular flood required a storage capacity of 202,000 acre-ft.

In the foregoing example (which is typical of reservoir design floods), there is no period other than the full duration of the design flood that can be taken logically as the length of the equivalent rectangular flood. That period is too long, however, and its use does not provide sufficient storage. The time may be diminished continuously with increasing requirements for storage, but there is no way of determining the correct volume without routing through the reservoir. There seems to be no logical point on the hydrograph, therefore, at which the length of the equivalent rectangular flood can be fixed.

In certain cases it may be desirable to reverse the procedure of the foregoing example. The volume of storage available may be fixed, instead of the permissible outlet discharge, and the problem is then to determine the maximum reduction. This problem also requires the calculation of the quantity x; hence, the difficulty of the equivalent rectangular flood is met again.

It is highly desirable to start from the known factors of a problem and to work progressively toward a solution without resorting to trial methods; but it is apparent that the difficulty of the equivalent design flood prevents such a course in analyzing flood storage capacity with the authors' method.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

DISCUSSIONS

AN IMPROVED METHOD FOR ADJUSTING LEVEL AND TRAVERSE SURVEYS

Discussion

BY ROBERT C. SHELDON, ASSOC. M. AM. SOC. C. E.

ROBERT C. SHELDON,[16] Assoc. M. Am. Soc. C. E. (by letter).[16a]—As progress in engineering and surveying increases the need for, and use of, coordinate systems of surveying, in which all traverses are closed on points with fixed positions, the need for a satisfactory method of adjusting the closing errors of the traverse nets is correspondingly increased. The method presented in the paper fills this need admirably. It was the writer's first thought that any method that used the typical Doolittle method for the solution of the necessary simultaneous equations would be too difficult for the average engineering office to use. The authors, however, have so simplified the preliminary work of forming the equations as to make the method easy to use by any office with a computing machine.

The writer has applied the method on a traverse net that had previously been adjusted by another method. The net contained five fixed stations and eight interior junctions. The previous adjustment had been by what might be called "the single-line" method, or "path-of-least-resistance" method. That is, the field data between two fixed stations were computed into latitude and longitude increments in feet, after which the errors of closure at the second fixed station were distributed over the traverse in proportion to the lengths of the courses. After this distribution, the points on the line were considered as fixed stations and any lines starting or ending on them were adjusted to close.

The fixed points were triangulation stations that had been placed and adjusted so that, without question, their accuracy was of a higher degree than 1 : 10,000. The field methods used in running the traverses were such that accuracies of 1 : 10,000 could have been expected and 1 : 5,000 was required. The lines running between the fixed stations were well within the requirements, but several of the ties between the interior junction points were not.

NOTE.—This paper by Clarence Norris, Esq., and Julius L. Speert, Assoc. M. Am. Soc. C. E., was published in October, 1939, *Proceedings*. Discusson on this paper has appeared in *Proceedings*, as follows: January, 1940, by Messrs. George H. Dell, and Howard S. Rappleye; and February, 1940, by Messrs. R. M. Wilson, Cleveland B. Coe, and George D. Whitmore and C. C. Miner and W. O. Byrd.

[16] Asst. Engr., Section of Surveys, The Panama Canal, Balboa, Canal Zone.

[16a] Received by the Secretary January 31, 1940.

The corrections to the traverses, to make them close on the fixed points, or junction points, by the single-line method were: Minimum 1 : 9,000, maximum 1 : 3,000, and the average over the net 1 : 6,000. After adjustment by the authors' method, the corrections to the traverses to make them close on the adjusted junction points were: Minimum 1 : 20,000 (except one line practically precise closure), maximum 1 : 5,000, and average over the entire net 1 : 11,000. The adjustment was made using the field values and not those found by the previous adjustment.

The time required in making the adjustment by the authors' method was a matter of hours. With experienced men, this adjustment on a net of this size would seldom require more than two days. The time lost in unsuccessful attempts to find errors in the field and office work on the traverses that did not close, using the single-line method, was a matter of several days.

The writer is in complete agreement with the conclusions of the authors and believes that the method presented will be a valuable aid to any engineering or surveying organization using a coordinate method of surveying. The writer's application of the method was made on the Panama Canal system using degrees, minutes, and feet of both latitude and longitude rather than the northing and eastings used by the authors.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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DISCUSSIONS

PROBLEMS AND TRENDS IN ACTIVATED SLUDGE PRACTICE

Discussion

By Langdon Pearse, M. Am. Soc. C. E.

Langdon Pearse,[3] M. Am. Soc. C. E. (by letter).[3a]—A useful compilation of statistical data on existing activated sludge works is presented in this paper, but it falls short of picturing the art accurately and of giving a weighted appraisal of current values in its practice. Furthermore, the author makes suggestions, without comment, for trial. With this preamble, a few comments may be helpful.

North Side Works.—The North Side Works of the Sanitary District of Chicago is cited as an example of treating a sewage flow considerably in excess of its design capacity. The plant, however, was planned hydraulically to care for a considerable excess flow over the capacity assigned in 1928, so that by the addition in 1937, of a few final settling basins, a plant rated in 1928, on average capacity, at 180 mgd is now rated at 250 mgd. As this was the first plant ever constructed to handle an average flow of more than 180 mgd, the various parts were liberally designed to permit, later, the demonstration of possible economies which were hoped for, although not then established by current practice. Thus a nominal 6-hr aeration period was reduced to 5 hr, and for more than one year one battery (of three) operated at a 3-hr period (see Table 7).

Grit Chamber.—With the adoption, by the Sanitary District, of dewatering and incineration of sludge, the grit chambers were omitted at the Calumet and Southwest works because it was thought that the grit could be handled in the preliminary tanks. At the Calumet works, the preliminary tanks were designed on a 10-min period, but actually have been working at periods of about 20 min. However, the plants, as constructed, include a hydraulic gap for the grit chambers. Operation at the Calumet works has demonstrated the need of a grit chamber under Chicago conditions, particularly for use during storms. In 1939, grit chambers were installed at the Calumet works on the sludge

Note.—This paper by Robert T. Regester, M. Am. Soc. C. E., was published in November, 1939, *Proceedings*. This discussion is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion of the paper.

[3] San. Engr., The San. Dist. of Chicago, Chicago, Ill.

[3a] Received by the Secretary February 1, 1940.

removed from the preliminary tanks—1 mgd for each of two units. They are cleaned mechanically, with flexibility in feed and in varying effluent weir length.

Sludge Handling.—Whether the method of sludge handling by the Sanitary District is correctly described as a "use of waste heat" is a question, inasmuch as there is practically no waste heat in the dewatering and incineration of sludge in the District under normal conditions. The heat recovered by burning the sludge (when burned) is used in drying the wet cake. The generation of steam depends primarily on the use of additional coal. To avoid odors, the gases to be vented to the atmosphere are passed through a heat zone of 1,500° F. Thus it was logical design for a large project like the Southwest Works to run the blowers, pumps, and electric generators by steam turbines, and make the boiler fire-box a means of deodorizing. Heat-dried sludge to be sold as fertilizer requires about one half ton of coal to dry one ton of sludge. The economy in operation is not primarily derived from heat recovered as a by-product, but from consolidating the heat-drying and boiler operations under a common operating crew.

TABLE 7.—NORTH SIDE SEWAGE TREATMENT WORKS AERATION PERIOD AND AIR CONSUMPTION,* 1937

Month	Aeration Period, in Hours			Quantity of Air, in Cubic Feet per Gallon of Sewage			Final Effluent 5-Day B.O.D., in Parts per Million		
	A	B	C	A	B	C	A	B	C
January	4.7	2.8	4.9	0.28	0.28	0.31	10.9	12.1	11.8
February	5.0	3.1	5.1	0.38	0.35	0.35	10.8	12.0	9.9
March	5.4	2.9	4.5	0.43	0.36	0.38	9.8	11.3	9.6
April	4.5	2.5	4.1	0.31	0.29	0.30	7.1	8.7	7.4
May	4.2	2.6	4.0	0.34	0.33	0.35	8.0	8.7	9.0
June	4.6	2.9	4.7	0.39	0.38	0.38	5.8	8.4	6.9
July	5.0	3.2	4.5	0.38	0.40	0.36	6 5	7.1	7.9
August	5.0	2.9	4.3	0.39	0.39	0.36	7.4	6.6	7.1
September	4.4	2.9	4.1	0.37	0.38	0.34	9.5	8.4	11.2
October	4.5	2.9	4.5	0.31	0.35	0.34	7.2	7.2	7.4
November	5.3	3.1	4.5	0.38	0.33	0.33	10.8	10.2	9.7
December	6.0	4.2	5.3	0.40	0.34	0.35	14.4	13.6	12.4
Average	4.9	3.0	4.5	0.36	0.35	0.35	9.0	9.5	9.2

* A, B, and C refer to Batteries A, B, and C.

Preliminary Treatment.—The use of fine screens should have been noted for preliminary treatment, particularly for a situation like that at Milwaukee, where all visible solids of sewage origin were kept out of the lake. At Chicago, preliminary settling was installed at the North Side, originally, to reduce the volume of sludge pumped through a 17-mile pipe line and, by removing a part of the coarser solids, to reduce the quantity of air required. Actual tests on a plant scale, however, indicate that somewhat less air is required. Preliminary treatment is necessary on such a weak sewage, although probably not in proportion to the solids removed. Also, preliminary settling was used at the Calumet and the Southwest works because large-scale tests demonstrated that a mixture of fresh and activated sludge dewatered more readily on a vacuum filter than activated sludge alone.

Aeration.—Under "aeration," there are a number of points worth consideration. As to aeration period, on a weak sewage at Chicago (West Side, settled), in experimental work, the aeration period was reduced to less than 2 hr before the quality of the effluent began to deteriorate. In a number of installations in the United States, many devices or arrangements have been tried, such as introducing settled sewage at three points in an aeration tank, after the introduction of return sludge, and graduated or tapered aeration. As yet the value of such procedure has not been demonstrated.

Aeration Tank Design.—Cross baffles have been used in Great Britain to a limited extent, and were installed at Pasadena, Calif., many years ago. Later they were used at Tenafly, N. J., and Hagerstown, Md. In the design of the Sanitary District plants, long runs have been provided in the aeration tanks.

Diffuser Plates.—An important trend, omitted by Mr. Regester, should be noted in the rating of diffuser plates. Early installations used porous plates with a porosity around 9 to 18 cu ft of air per min per sq ft at 2-in. water pressure (Milwaukee, 9 to 13; North Side, Chicago, 14 to 18). Since 1929, the tendency has been toward more porous plates at Indianapolis, Milwaukee, Chicago, and elsewhere. Those purchased for the Southwest (in 1936) and Calumet works (in 1934) ranged from 36 to 44 cu ft per min. These higher porosities are favored on the theory of lower friction losses and reduced clogging. However, more than 90% of the original plates installed in the North Side Works in 1928 were still in service 11 years later, with comparatively slight increments in pressure loss. However, at Chicago, sudden clogging has occurred in experimental work on tannery wastes due to the deposit of calcium carbonate on the plate surface, and in operating the new works at Calumet, clogging was due to ferric oxide deposit on the surface of the plates, formed from spent pickling liquors high in ferrous iron which entered the sewage.

Flow Regulation.—At the North Side and Calumet works, the sludge flow from individual settling tanks is controlled manually, by the setting of a gate valve at the former plant and by the overflow of a fountainhead regulator at the latter. No venturi rate controllers are used. At both plants, the sludge from the final settling tanks is discharged into a common channel in each battery, and thence to the return-sludge pumping station, where a return-sludge pump for each battery, plus one spare pump, is provided. The pumps are driven by variable-speed motors. Originally the speed of these motors was controlled by electrolytic controllers, actuated by a float in the pump suction well. However, at the North Side, these have been replaced by manually-operated resistance-type controllers. At each plant the return-sludge flow from each battery is metered and then passed to the sewage conduit leading to the aeration tanks. The mixed-liquor flow from each aeration tank is metered at both plants.

At the Southwest works, the sludge is removed from each final settling tank by a pair of air lifts, the sludge from all tanks in each battery discharging into a common channel leading to the mixing channel. The return-sludge flow from each battery is metered. The quantity of sludge from individual tanks is estimated from the air used by the air lifts. The air to the aeration tanks is regulated by means of globe or angle valves at all three plants. The flow to

each half tank at the Southwest works, and the total flow to each battery of tanks, are metered.

The use of rate controllers on mixed-liquor flow was canvassed years ago at Chicago, but was discarded as being somewhat too refined.

Operating Gallery.—The utilization of an operating gallery in activated sludge practice was developed by the writer in 1920, from his earlier experience in designing rapid water filter plants. In large plants, particularly in northern climates, there is a need for operating galleries. The housing has been reduced in size at Calumet and Southwest. At the North Side, a number of meters and recording devices that were installed originally have been abandoned, after extended trial.

Air Cleaning.—The cleaning of air before compression is important. Water washing has been found to be of little value. Oil-coated media used as a filter are commonly favored, whether in fixed frames or moving continuously, being dipped in an oil bath.

Blower Installation.—The design of a blower installation depends on the size of plant and type of blower. Where centrifugal blowers are used, the use of a blow-off is almost unknown. With Connersville or "tub" types, a blow-off may be highly desirable.

Final Sedimentation.—Final sedimentation tanks have increased in size since the San Marcos plant was put in operation in 1915. The need of weir study on activated-sludge final settling tanks was first stressed at Chicago and Springfield, Ill. Actual tests were made by S. I. Zack, M. Am. Soc. C. E., under the writer's direction about 1931 at the Des Plaines River Treatment Works of the Sanitary District. The H-weirs and end weirs set back from the end wall were developed at that time. Supplemental weirs or gutters had been installed as early as 1919 at Houston, Tex., by G. L. Fugate. Tests on the original North Side design have shown that the final settling tanks are satisfactory, with flow admitted to a square tank from two opposite sides, the effluent being taken over the weirs of three cross troughs. Before the Calumet plant was designed, S. L. Tolman made an extended series of tests at Chicago, Milwaukee, and San Antonio, Tex., for the Sanitary District, to determine allowable rates of flow.

Various devices to determine levels of the sludge blanket have been made. So far as the writer knows, the first use of the photo-electric cell was at the Des Plaines River Works in 1926. This device was designed to give an alarm if the sludge level rose too high. Later, attempts were made to connect the cell to control devices. From a practical standpoint, the use of small spouting air lifts reaching to different depths has proved satisfactory in all the plants of the Sanitary District.

Sludge Disposal.—In the question of sludge disposal, W. L. Stevenson, M. Am. Soc. C. E., first indicated the possibility of digesting fresh and activated sludge in an Imhoff tank.[4] So far as the writer knows, the first use of this procedure was in the Calumet plant of the Sanitary District, in 1922. For the small plants, the digestion of mixed sludge offers a suitable method of handling the sludge. For the large plants, like the Calumet, Southwest, and North Side

[4] *Proceedings*, Am. Soc. of Municipal Improvements, 1916, pp. 460–463.

of the Sanitary District, other procedures were also studied, resulting in the development of the dewatering and flash-drying system, with incineration or disposal of the heat-dried sludge for fertilizer. Since the Southwest works began production of heat-dried activated sludge in the autumn of 1939, orders for more than 39,000 tons had been booked for future delivery, prior to June 30, 1940. The Milwaukee production was "sold out" for the same period. The first 6,700 tons delivered from the Southwest works averaged about 6.46% ammonia, and grossed $8.07 per ton, f.o.b. Chicago.

Conditioning of Sludge.—The conditioning of activated sludge for vacuum filtration was made practical by the discovery, at the old Calumet works, of the value of ferric chloride. Through the cooperation of the chemical industry, the price of ferric chloride has been reduced steadily. Bids on considerable tonnage received in 1939 show a low of 1.21 cents per lb of anhydrous ferric chloride, or with freight on water included, approximately 1.8 cents, delivered. This is a remarkable reduction from about 8 cents per lb current prior to 1925.

Centrifuges.—The application of centrifuges to dewatering sewage sludge has proved a "headache" because of the difficulty of obtaining adequate removals of solids at a reasonable cost. The Sanitary District made tests about 1932 on a sufficient scale to demonstrate the requirements to produce the desired results in removal of solids. Working plans were made for practical units, but a machine was never built because the procedure was too expensive. However, centrifuges of batch laundry type are used to dewater screenings at Milwaukee.

Final Filtration.—For further treatment, the value of filtration following final sedimentation has yet to be demonstrated as a general procedure.

Seasonal Operation.—The Indianapolis plant for many years has been operating on a basis of a seasonal operation, whereby in the colder months the major part of the plant uses aeration without sludge return, only a minor portion having sludge return. From June to October as much sewage as practicable is given complete treatment, the amount varying with the conditions each year.

Starting of New Plants.—Probably the first of the largest plants in the United States was the Houston South Works, started in 1917, following the discovery by Edward Bartow, M. Am. Soc. C. E., and F. W. Mohlman, at the University of Illinois[5] in 1915, that an activated sludge plant could be started off on a continuous basis, and sludge developed in 10 to 15 days showed marked purification. This eliminated the use of a fill and draw or batch process. So far as the writer can ascertain, all the later works of any material size started off along the lines laid down by Messrs. Bartow and Mohlman. Possibly the use in Ohio, by small plant operators, of the fill and draw process for starting may be justified, but the reasons for such procedure should be made clear.

It is hoped that Mr. Regester will report what Mr. Donaldson accomplished by a proposed procedure to reduce the mixed liquor solids from 1,500 ppm to 1,300 ppm. It seems questionable whether such a change would make an appreciable difference in operating results.

Effect of Depth on Concentration of Sludge.—In the discussion of the effect of depth on concentration of activated sludge, Mr. Regester states: "It is

[5] *Bulletin No. 14*, Univ. of Illinois, Urbana, Ill., pp. 325–335.

probable that increased depth of sludge blanket, with consequent compaction, produces a greater density of the sludge." A few tests, made in 1931 by the Sanitary District, on different depths of tanks indicate roughly that the density of activated sludge may vary but little with depths from 10 to 30 ft. For that reason, various devices were explored to aid concentration. A picket fence concentrator (Dorr type) was tried at the Des Plaines River Plant in the early fall of 1931.

Industrial Wastes.—The reference to the application of activated sludge to industrial wastes is necessarily sketchy. The Sanitary District has operated testing stations on packing house, corn products, and tannery wastes. Others have given much study to different industries. Out of this study (which began as early as 1916) has grown a feeling that treatment of industrial wastes mixed with domestic sewage in a municipal plant may prove a practical solution of the problem in many situations. In the case of the corn products industry, bottling up the waste waters and concentration of solids, with removal by vacuum filtration, together with other operating adjustments in the manufacturing procedure, proved a very practical solution.

Sampling.—One important feature of plant practice, notably for activated sludge, but also all sewage works operation, is adequate and reliable sampling. From an experience in the work on corn products waste in 1920, the Sanitary District has developed a practice of using automatic samplers as far as practicable. The earlier samplers were home-made. Later types were developed by ingenious manufacturers.

Desiderata.—The list of fifteen desiderata, which Mr. Regester catalogs, includes many on which a considerable fund of information is available. Much of this information has been collected by operating workers from actual practice, and has been described in part in published articles and annual reports. Some of the questions he raises involve not only activated sludge, but the entire field of sewage treatment and industrial waste handling. From more than 30 years' experience with The Sanitary District of Chicago, the writer is firmly imbued with the idea of following up works operation carefully, making large-scale tests as far as practicable, and determining the relative value of various procedures, as to end results, costs, and efficiency. If the designer will seek the criticism and comment of the operating staff, better designs and more economical operating conditions should result.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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DISCUSSIONS

BRIDGE AND TUNNEL APPROACHES

Discussion

By George Hartley, Esq.

George Hartley,[10] Esq. (by letter).[10a]—Problems related to the proper design of bridge and tunnel approaches are increasing rapidly with the present extensive use of toll charges on highways in the United States. Typical construction is exemplified in the present trend toward toll express highways and parkways as well as the steady increase in the number of toll bridges and tunnels. In his excellent paper, Mr. Curtin has summarized many of the conditions and problems related to the present major toll bridge and tunnel construction. Some attention should be given to the great percentage of toll structures on which the toll collection requirements are for only a small volume of traffic which, in general, does not approach the traffic volume attained by the structures analyzed by Mr. Curtin.

An obsolete type of toll booth is shown in Fig. 8, at which two-way toll is levied by a single collector. A number of these structures are still in existence in rural low-traffic areas and are "left-overs" from the day of the horse and buggy. The toll gate placed across the highway required all vehicles to stop; and the barrier was raised after the toll had been collected. In bad weather the toll collector is exposed to the elements, and at all times there is the danger, during heavy traffic periods, of being run down by vehicles when the highway is crossed to collect tolls.

This discussion centers about the relative traffic-volume capacity of the various toll plaza arrangements considered. The basic assumptions upon which these evaluations were made are as follows: A collection time of 6 sec is assumed as a constant factor when toll collection is made by the driver of the vehicle out of the left-hand or driver's window; when the driver is required to reach across the front seat to pay toll, a collection time of 8 sec has been assumed. These assumed applications of time are for use in relative traffic-capacity considerations and are based on the 6-sec to 8-sec collection periods given by Mr. Curtin as representative of the conditions at the George Washing-

Note.—This paper by John F. Curtin, Jun. Am. Soc. C. E., was published in November, 1939, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: February, 1940, by Messrs. Dean G. Edwards, and John W. Beretta.

[10] Engr. with Robinson and Steinman, Cons. Engrs., New York, N. Y.

[10a] Received by the Secretary February 6, 1940.

ton Bridge and the Golden Gate Bridge. The 2-sec differential is substantiated in a measure by Glenn B. Woodruff, M. Am. Soc. C. E., in his observation of traffic conditions that exist at the toll house of the Trans-bay Bridge (corre-

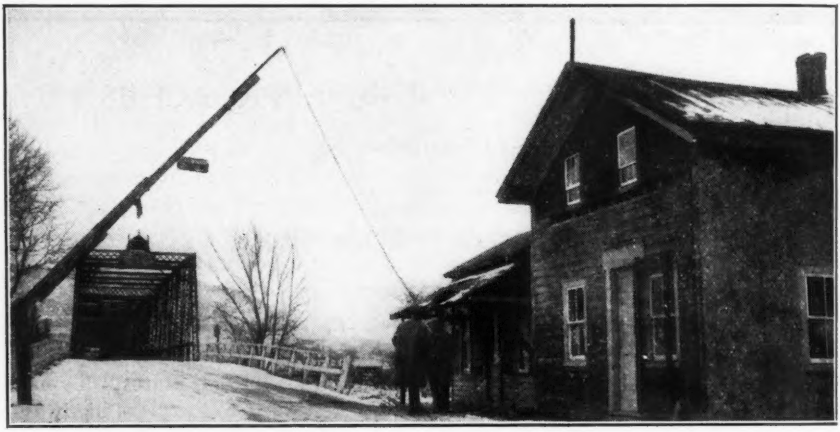
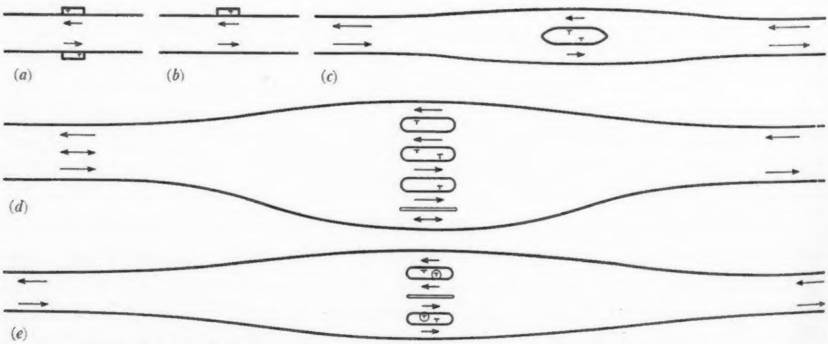


FIG. 8.—TOLL HOUSE OF THE SMITHBORO BRIDGE COMPANY, SMITHBORO, N. Y.

spondence with the writer). At this crossing traffic that pays toll from the driver's side of the vehicle moves freely, whereas in the adjacent lane, which requires the driver to reach across the front seat in paying toll, an appreciable lag has been noted.



(a) 225 to 450 Vehicles per Hr
(b) 450 Vehicles per Lane per Hr
(c) 600 Vehicles per Lane per Hr
(d) 1,050 Vehicles per Lane per Hr
(e) 1,800 Vehicles per Lane per Hr

FIG. 9.—TOLL-BOOTH LAYOUTS, SHOWING MAXIMUM CAPACITIES

Applying the 8-sec collection factor to the arrangement shown in Fig. 9(*a*), the maximum traffic that this type of structure is capable of handling covers a range of from 225 to 450 vehicles per hr (see Fig. 9(*a*)). Even the value of 225 vehicles per hr may be excessive as the collector must go to the center of the

highway in order to collect toll from traffic in the far lane which, in other types, would require only a 6-sec collection time. The time to cross the highway may be considerably more than 2 sec.

With the development of the motor vehicle, and the subsequent increase in highway traffic, another toll house was added as illustrated diagrammatically in Fig. 9(*b*). Thus tolls were collected separately for each traffic lane and the maximum traffic that could be handled was 450 vehicles per lane per hr. In this type the collection of toll is not convenient as the operator is still required to reach across the front seat in order to pay toll. A typical installation of this type is shown in Fig. 10. Traffic keeps to the left. Drivers seated on the left side of the vehicle can pay toll conveniently; and a maximum traffic capacity of 600 vehicles per lane per hr is possible. As most vehicles in Australia are operated with the driver on the right side of the vehicle there is little or no increase in traffic capacity over the arrangement shown in Fig. 9(*b*). The total capacity range under operating conditions shown in Fig. 9(*b*) will range from 225 vehicles per hr with one toll collector on duty to a maximum traffic capacity of 900 vehicles per hr when two toll collectors are on duty.

In Fig. 9(*b*) two toll collectors would be required for normal operating conditions, and in localities where traffic volume is small an added expense is incurred. This condition may be remedied by placing the toll booth in the center of the highway, as shown in Fig. 9(*c*), so that the drivers in both lanes are placed for convenient toll collection with a resulting traffic capacity of 600 vehicles per lane per hr when two toll collectors are operating. As stated, this form is the best for low traffic conditions as one collector is able to collect tolls at a rate that approaches 600 vehicles per hr, and it is only necessary to employ

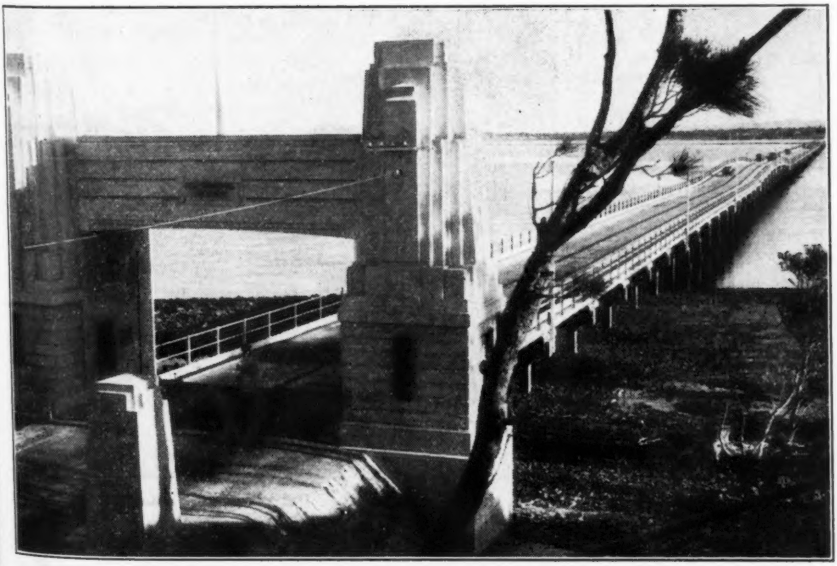


FIG. 10.—TOLL HOUSE FOR THE HORNIBROOK BRIDGE, AUSTRALIA

two toll collectors during peak traffic conditions—that is, as the total traffic approaches 600 vehicles per hr.

As traffic volume increases beyond 600 vehicles per lane per hr but does not exceed 1,050 vehicles per lane per hr an additional toll-collecting lane is required,

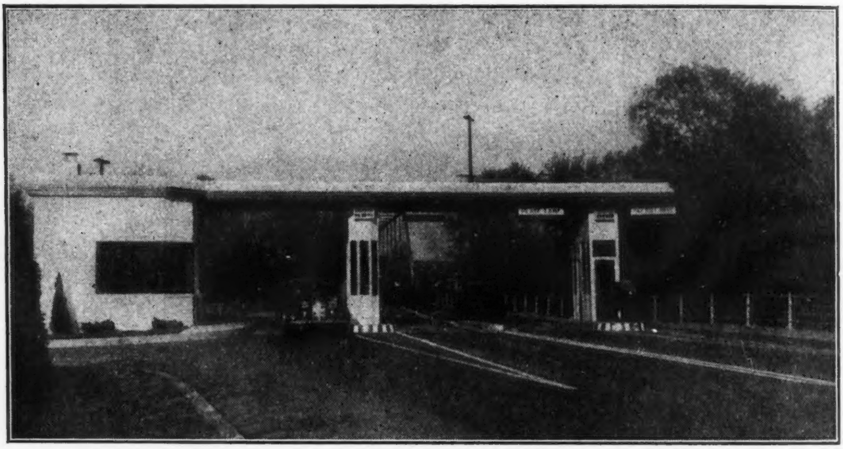


FIG. 11.—TOLL HOUSE OF THE WALNUT STREET BRIDGE, HARRISBURG, PA.

FIG. 12.—TOLL HOUSE OF THE MOUNT HOPE BRIDGE, RHODE ISLAND

the schematic arrangement of which is shown in Fig. 9(*d*). In this diagram the encircled toll collection symbol indicates the absence of toll collectors at these points for an installation made by the Walnut Street Bridge at Harrisburg, Pa.

(Fig. 11). Passengers are allowed commutation rates and a large percentage of the total traffic use them. Special plates, in effect for one month, are issued and displayed by the vehicles, which are then not required to stop at the toll booth.

With additional increases in traffic to more than 1,050, but not exceeding 1,650, vehicles per lane per hr, additional toll collection facilities must be provided; and, with the type illustrated in Fig. 9(*d*), an additional toll collecting lane must be provided for every alternate increase in traffic of 450 and 600 vehicles per lane per hr. Under the condition of maximum design of 1,650 vehicles per lane per hr the layout in Fig. 9(*d*) is extended to accommodate an additional toll booth which is placed on the center line of the highway so that two lines of traffic traveling in opposite directions are separated for toll collection. This condition is illustrated by Mr. Curtin in Fig. 2, for the installation at the Trans-bay Bridge. Under this condition, there are three toll-collecting lanes capable of handling 1,650 vehicles per lane per hr, and the author's statement that three toll-collecting lanes are required to pass 1,500 vehicles per lane per hr is substantiated. At the annual meeting of the Highway Division in New York, N. Y., on January 18, 1940, Guy Kelcey, M. Am. Soc. C. E., quoted actual highway traffic conditions in which traffic volume was approximately 1,800 vehicles per lane per hr at an estimated speed of 35 miles per hr. Sigvald Johannesson, M. Am. Soc. C. E., has given an equation for maximum density of traffic which, at vehicle speeds of 35 miles per hr and 40 miles per hr, gives maximum values of 1,812 and 1,869 vehicles per lane per hr.[11] These values are in excess of the author's design value of 1,500 vehicles per lane per hr, and four toll lanes would be required in the extension of the design in Fig. 9(*d*) making possible the collection of tolls for traffic as great as 2,100 vehicles per lane per hr. To pass the foregoing maximum values of traffic without resorting to the use of additional lanes the design given in Fig. 9(*e*) may be adopted which, on the basis of the previous computations, is capable of passing a volume of 1,800 vehicles per lane per hr in three lanes. This theoretical value may be reduced slightly as there is an increase in the horizontal travel of the vehicles across the plaza. An actual installation of this type is shown in Fig. 12 at the Mount Hope Bridge in Rhode Island. In this installation every possible condition of toll collection with respect to varying volume can be accommodated. Fig. 9(*e*) represents the schematic arrangement of this toll booth. The bottom

FIG. 13.—SUGGESTED ARRANGEMENT OF TOLL BOOTHS TO ALLOW GREATEST FREEDOM TO TRAFFIC MOVEMENT

[11] "Highway Economics," by Sigvald Johannesson, 1st Ed., McGraw-Hill Book Co., Inc., New York and London, 1931, p. 90.

lane, which indicates travel in both directions, was built to accommodate vehicles having unusual clearance requirements.

In closing the writer would like to call attention to Fig. 13 which shows an arrangement for the construction of toll booth plazas that has unusual merit. The toll booths are separated as well as staggered and the diagram illustrates the application to modern express highways where a central parkway strip is used. This solves the problem with regard to maximum-flow capacity in which particular stress is placed on the location of the entrance to the plaza with the criterion that no abrupt changes can occur in the direction of the traffic flow. As shown in Fig. 13, the entrance to the plaza is centered about the toll booths and under low-traffic conditions traffic keeps to the right in approaching the booths. After toll has been collected the driver of the vehicle is not required to travel transversely as he is in line with the continuation of the highway and drives straight ahead. Many advantages are obtained with this design: The horizontal travel of vehicles is reduced to a minimum; for parkway construction the toll booth plaza is kept within the roadway width; the right-of-way purchase is reduced; and, for the condition illustrated, no additional right of way is required. In areas of high realty costs this may result in large savings. The arrangement accommodates traffic volumes as great as 1,050 vehicles per lane per hr; and, with increase in capacity, construction will extend on to the shoulders. For the design in Fig. 9(*d*), peak traffic conditions during the day, with practically all the travel in one direction, may be accommodated by utilizing the toll booths. The latter normally would be used for traffic traveling in the opposite direction, and this is not possible with the design in Fig. 13 as there is an actual segregation of traffic.

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DISCUSSIONS

TREND IN HYDRAULIC TURBINE PRACTICE A SYMPOSIUM

Discussion

BY MESSRS. LEWIS F. MOODY, AND R. E. B. SHARP

LEWIS F. MOODY,[24] ESQ. (by letter).[24a]—A wide range of subjects is treated in the interesting paper by Mr. Winter. The writer will confine his discussion to only one of them—centrifugal pumps operating as turbines.

In 1931 and 1933, models were tested in the I. P. Morris laboratory embodying certain ideas of the writer (see U. S. Patent 1,919,376 for a detailed description of the principles applied) for the construction of a reversible pump-turbine unit suitable for pumped storage plants. The runners assumed forms intermediate in proportions between pump and turbine practice. As Mr. Winter has found in the tests he mentions, surprizingly high turbine efficiencies were obtained; and the teachings of these experiments has had a material effect on subsequent practice in turbine design. The models tested were complete turbines with wicket gates and draft tubes. Fig. 19 shows the results obtained in the laboratory on these models.

In comparing the results of the reversed operation of pumps, cited by Mr. Winter, with turbines of normal design, several important factors should be considered. First, the efficiency as a turbine needs to be discounted somewhat due to the absence of a draft tube. The efficiencies based on net head on the machine itself, exclusive of velocity head in the discharge pipe, will be reduced when charging the unit with the duty of regaining the velocity head in the pipe, the function performed by the draft tube. This correction under the conditions of most of these tests at the Pasadena laboratory will be of the order of a little more than 0.5%, which is not great but has some effect on the comparison.

A more important factor is a consequence of the abnormally small eye or throat diameter of pump impellers of the best design. For turbine specific speeds within the range of about 25 to 30, pump impellers which will give these specific speeds will have an eye diameter of about seven tenths to eight tenths

NOTE.—This Symposium was published in November, 1939, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: January, 1940, by Messrs. W. S. Pardoe, and Donald H. Mattern.

[24] Prof. of Hydr. Eng., Princeton Univ., Princeton, N. J.

[24a] Received by the Secretary February 2, 1940.

of the throat diameter of normal turbines of the same specific speeds. When it is taken into account that the velocity head at the throat varies inversely as the fourth power of the diameter, it is found that the reversed pump will have from two and one half to four times the throat velocity head of the turbine of normal design. Naturally, this greatly increases the cavitation factor of the reversed pump, as compared to a normal turbine, and limits it to installation requirements which could be met by normal turbines approaching perhaps double its specific speed.

Still another factor is the abnormal proportions required of the turbine due to the large outside diameter of the pump impeller when compared to a normal turbine. Within the range of specific speeds mentioned, the reversed pump impeller will be about 20% larger in diameter than a turbine of the best normal design. Since the flow in the casing must encounter stay vanes and wicket

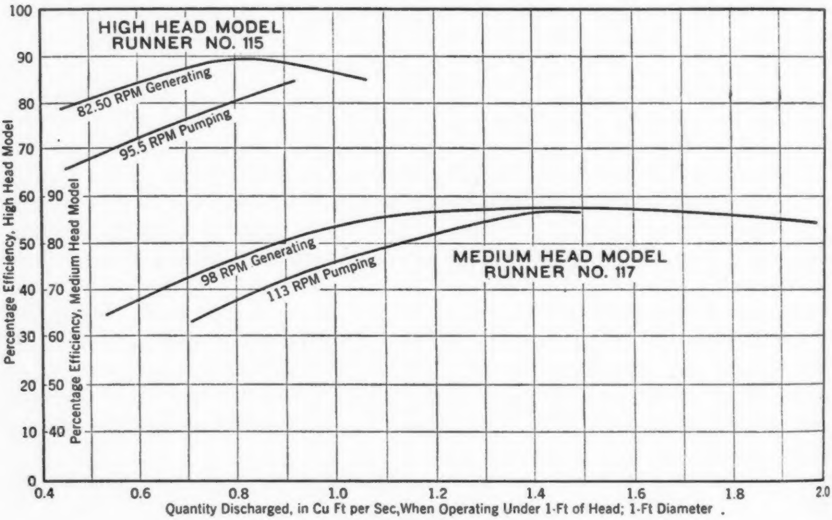

FIG. 19

gates, it is not advisable, in a turbine, to use casing velocities approaching those in pump volutes, or appreciably higher than now normally used. Consequently the over-all sizes of the turbine will be increased materially if the proportions of the pump impeller are adhered to, with a corresponding increase in cost. In a pump impeller operating as a pump, the large outside diameter is necessary to permit sufficiently gradual deceleration of the relative velocities; in a turbine runner there is no such requirement, since the flow is accelerating, and more favorable velocities result from a considerable reduction in diameter.

In comparing the effects of such factors, the only conclusive test would be to compare complete turbine models of normal and abnormal design under equal test conditions in the same laboratory, a procedure which has been the standard method of research in the development of the turbine to its present state.

Passing to the paper by Mr. Davis, the writer finds himself in such general agreement with this valuable paper that he will confine his discussion to a single point—the prediction of field efficiencies from model tests.

When the formula for "stepping up" efficiencies from model to prototype was first introduced, it was expressed in the form

$$\frac{1 - e}{1 - e_1} = \left(\frac{D_1}{D}\right)^p \left(\frac{H_1}{H}\right)^q \dots\dots\dots\dots\dots\dots(18)$$

with the exponents p and q to be determined empirically from all available test comparisons. The data then available seemed to indicate an exponent p very close to 0.25 for the diameter ratio, and a value for q of the order of 0.01. The effects of head, however, could be determined only roughly since no tests were available to show, directly, the effect of head variation on a given turbine. The expression was then used in this form (as given in Equation (9)) for several years. The writer felt, however, that there was not sufficient information available to support the exponent q used for the head ratio; and on theoretic grounds he decided to drop the head term completely—that is, to use the exponent $q = 0$; and when the formula was published in a paper presented before the Society in 1925,[12] the term was omitted intentionally, giving

$$\frac{1 - e}{1 - e_1} = \left(\frac{D_1}{D}\right)^{0.25} \dots\dots\dots\dots\dots\dots(19)$$

The reasons for omitting this term were the considerations that if turbine water passages can be classed properly as "rough conduits," under the designation given by L. Prandtl and Theodor von Kármán,[25] M. Am. Soc. C. E., then, at the high Reynolds numbers representing the turbine water passages, even in the models, viscous forces should be negligible and the losses should vary directly as the square of the velocity. It will be found, for example, that even in a 16-in. propeller turbine model, under a 3.5-ft head, the Reynolds number is of the order of 500,000.

Even with a very large variation of head it has been impossible to detect any effect of head. Thus, a model of the 35,000 hp Oak Grove turbine gave 87.6% efficiency under a head of less than 4 ft. When stepping up from the 12.065-in. diameter to the 50-in. diameter of the large unit, Equation (19) calls for 91.3% efficiency. The actual efficiency in the field, under an 850-ft head, was 91.5%.

Of course, there is nothing sacred in the numerical value of the exponent 0.25—this was intended to be determined from actual test results. The results secured in the I. P. Morris 16-in. models have been found to step up to field results by the use of this formula with as close agreement as could be expected; and satisfactory agreement has been reported from results in certain other laboratories. It is possible that the efficiencies given by the I. P. Morris 16-in. model tests may be somewhat low, as compared to some other laboratories, due to the effect of bearing friction on the low torque corresponding to the low head.

It should also be remembered that in these model tests the head is measured to tailwater without velocity-head correction, whereas the field tests used to

[12] "The Propeller Type Turbine," by Lewis F. Moody, *Transactions*, Am. Soc. C. E., Vol. 89 (1926), p. 625.

[25] "The Dynamics of Turbulent Flow," by Boris A. Bakhmeteff, M. Am. Soc. C. E., Princeton University Press.

determine the exponent were corrected for the velocity head at the draft tube discharge section. The A. S. M. E. testing code now (1940) in use corrects only for the velocity head in the tailrace. This factor has a material effect on propeller turbines, and tends to reduce the field efficiencies to an extent of approximately 1%. This consideration calls for a readjustment in the empirical exponent in the formula, based on a review of all data now available. Until such a study can be made, an exponent approaching one fifth, recommended by Mr. Davis, seems reasonable from present indications.

It should be realized that, in considering the question of the form of an efficiency step-up formula, conclusions can be reached only from a large mass of comparative test data. Single tests, naturally, will show material variations, since water measurement in large turbine tests is a difficult matter; other factors such as generator output are not always beyond question; exact similarity of model and prototype is not always secured; and uniform accuracy within even 1% error cannot be assumed. In the field of very large units few tests (not more than one or two) are available; and these are not sufficient, in the writer's opinion, to provide a basis for any general conclusions. It scarcely seems reasonable to expect any point of discontinuity, for example, beyond which the formula ceases to apply.

R. E. B. Sharp,[26] Esq. (by letter).[26a]—Referring to the heading "Turbines," the writer's experience regarding the shape of the throat ring is at variance with Mr. Winter's statement that "increased power and efficiency * * * may be obtained without the necessity of making the bottom of the throat ring spherical." Fig. 20 shows results of tests as to power, efficiency, and sigma break. The spherical throat ring is indicated by solid lines, and the straight throat ring by broken lines. These tests show that all three of these factors suffer relatively with the straight throat ring except at the highest value of ϕ where the maximum efficiency at that speed shows some improvement with the straight throat ring. However, the maximum efficiency obtained was secured at a lower value of ϕ with the spherical design. In other words, the writer has found that if the spherical throat ring is designed properly, the results secured are superior to those with the straight throat ring. With a properly designed spherical throat ring, the improvement secured with the reduced leakage loss through the clearance space more than offsets the additional burden on the draft tube due to the restricted diameter. Fig. 20 shows the design of a spherical throat ring used in obtaining these results. It is true that improperly designed spherical throat rings may be inferior to the straight throat ring, and where the curvature of the spherical throat ring is incorrect or abrupt there is a decided tendency for pitting to occur just below the restricted diameter.

Centrifugal Pumps as Turbines.—As turbines, these units have a distinct advantage over units designed as such, due to the absence of movable gates surrounding the runner. The reduced friction losses caused by the absence of these gates would tend to account for a high turbine efficiency of centrifugal

[26] Chf. Engr., I. P. Morris Dept., Baldwin Locomotive Works, Philadelphia, Pa.

[26a] Received by the Secretary February 6, 1940.

pumps; and absence of these gates prevents such units from having practicable value as turbines, even for pumped storage projects.

For a given value of turbine specific speed and capacity, a pump impeller has a materially larger over-all diameter than a turbine runner designed as such. The entire unit being built around this diameter, the efficiency advantage must be great to justify the additional cost resulting from the larger runner diameter.

Studies have been made and models tested in the United States in an attempt to develop a hydraulic unit for use both as a pump and, when run in

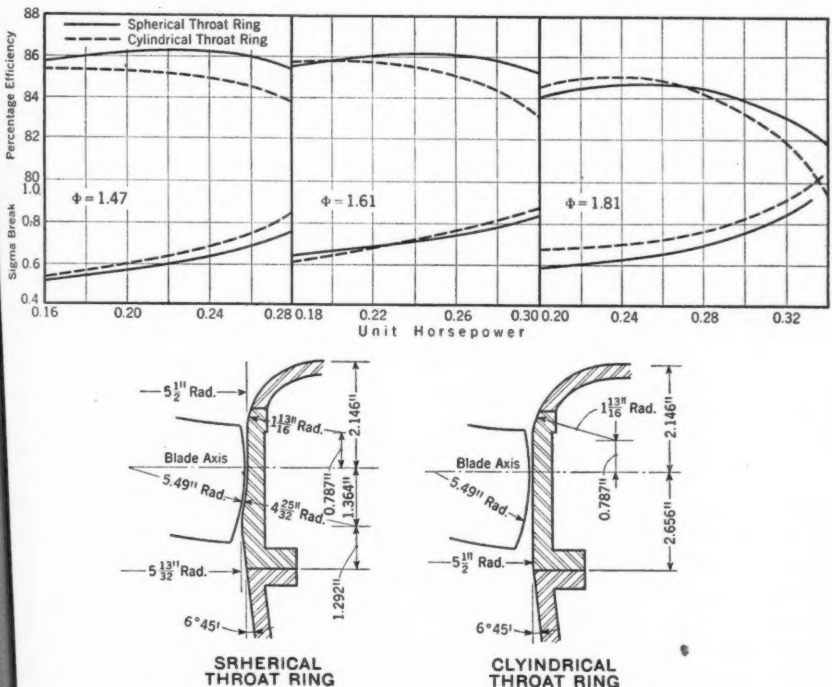

FIG. 20.—COMPARISON OF SPHERICAL AND STRAIGHT THROAT RINGS

reverse direction, as a turbine. These units are of the Francis type as the reservoir necessary usually prohibits a low-head pumped storage project. Very good efficiencies (89% when generating and 85% when pumping) have been obtained from these models, when operating both as a pump and as a turbine; however, a two-speed motor-generator is usually necessary where the unit takes the entire head both when pumping and generating.

When the head is greater than about 400 ft, present pump practice requires that more than one stage be used. On this basis, the arrangement shown in Fig. 21 has been devised. When generating, the pump-turbine takes the entire head, valve *A* is open, and valves *B* and *C* are closed. In order to pump, valve *A* is closed, valves *B* and *C* are open, and the booster pump is started,

thus priming the pump-turbine which is then started. When sufficient pressure is generated, the pump-turbine guide vanes are opened and flow up the penstock takes place. When generating, the relief valve is utilized in the normal manner for preventing pressure rise on rejection of load.

When pumping, this valve has an important function as a surge suppressor. It is essential, when this operation is taking place, that automatic means be provided for preventing excessive speed in the reverse of normal direction in the event of current failure. Under this condition the pump-turbine gates close upon current failure and the relief valve opens, thus preventing injurious water hammer due to the returning column of water in the penstock. The relief valve then closes slowly, permitting the column of water to come to rest at the desired rate. The closing of the pump-turbine gates prevents excessive reversed speed of that unit and also protects the booster pump downstream from it. The distinct advantage of this arrangement is that the head pumped against by the pump-turbine may be so selected that the revolutions per minute for each operation may be the same without loss of efficiency in either case. This is a material advantage from the standpoint of the cost of the motor-generator. This arrangement, furthermore, permits the booster pump to be placed well below tailwater (as may be desirable to give the desired value of sigma) and permits the pump-turbine to be placed above tailwater, both for the pumping and the generating cycle, with values of sigma which are adequate in each case.

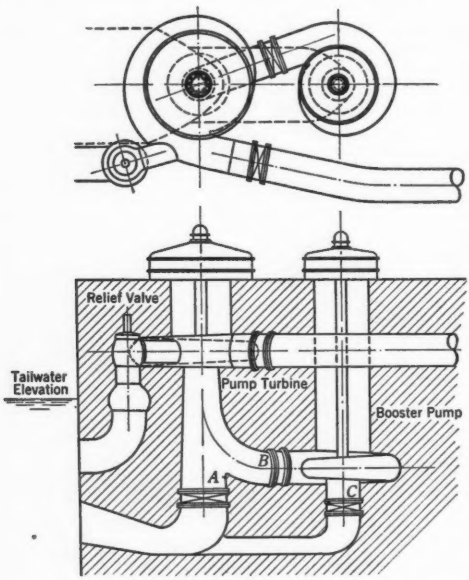

FIG. 21

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DISCUSSIONS

EFFECTS OF RIFLING ON FOUR-INCH PIPE TRANSPORTING SOLIDS

Discussion

By Fred R. Brown, Jun. Am. Soc. C. E.

Fred. R. Brown,[4] Jun. Am. Soc. C. E. (by letter).[4a]—Although the subject of rifled pipe is not entirely new, little has been done to investigate or make known its possibilities. As stated by the author, care must be taken in deciding whether there is justification for the use of rifled pipe, and in the determination of the specifications for the rifles, if such justification is found. Too many variables are present to justify any definite conclusions as to the specifications for the best type of rifling. For example, combinations of height and length of rifle can vary and yet give identical results. The matter of the velocity of the transported material is another item that will influence the specifications for rifling in pipes.

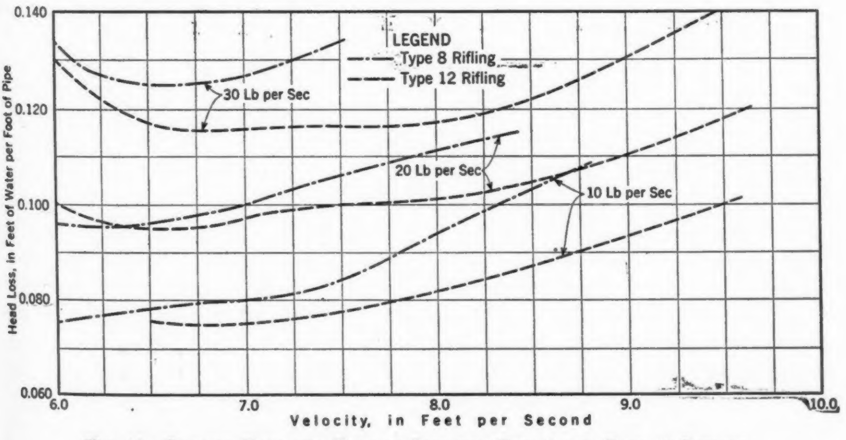


Fig. 10.—Loss of Head per Foot of Pipe for Type 8 and Type 12 Rifling

Note.—This paper by G. W. Howard, Jun. Am. Soc. C. E., was published in November, 1939, *Proceedings*. This discussion is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion of the paper.

[4] Junior Engr., U. S. Waterways Experiment Station, Vicksburg, Miss.

[4a] Received by the Secretary January 17, 1940.

The type 12 mixer (see Table 1), developed as a result of the first series of tests discussed by the author, was the most satisfactory for pipes transporting coarse material. Type 8, however, indicated better mixing qualities. The only difference in the specifications of the two was in rifle height; type 8 was $\frac{3}{4}$ in. high and type 12 was $\frac{1}{2}$ in. high. Decreasing the rifle height for equivalent lengths of rifling increases the efficiency of the pipe for concentrations of material above the critical zone in which the pipe may become blocked (see Fig. 10). For low velocities and high concentrations of material, however, the type 8 mixer in the pipe line permitted more material to be carried before blocking off than did the type 12 (see Fig. 11). Despite the fact that the type 8 mixer increased the mixing of the material, the great increase in overall efficiency, as a result of decreasing the rifle height, indicated that the type 12 mixer was the better design.

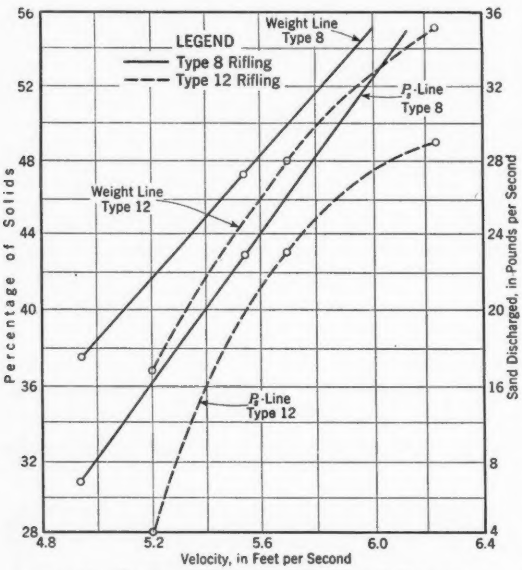
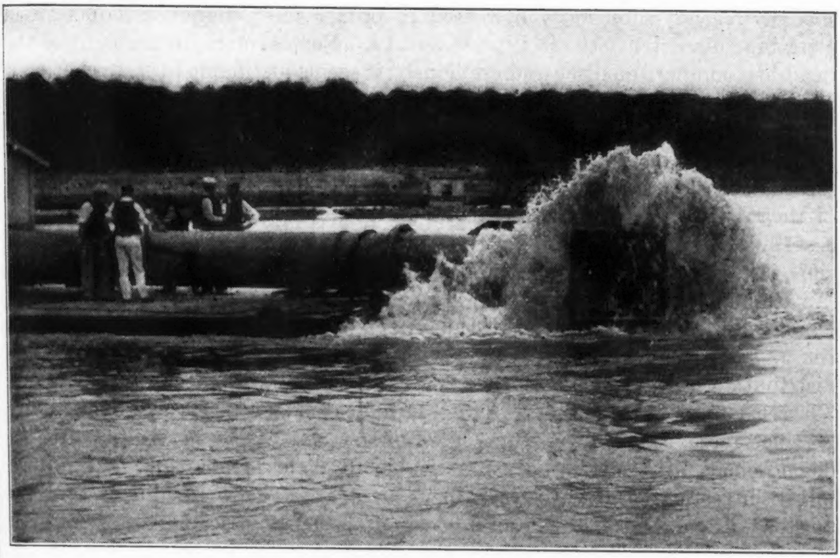
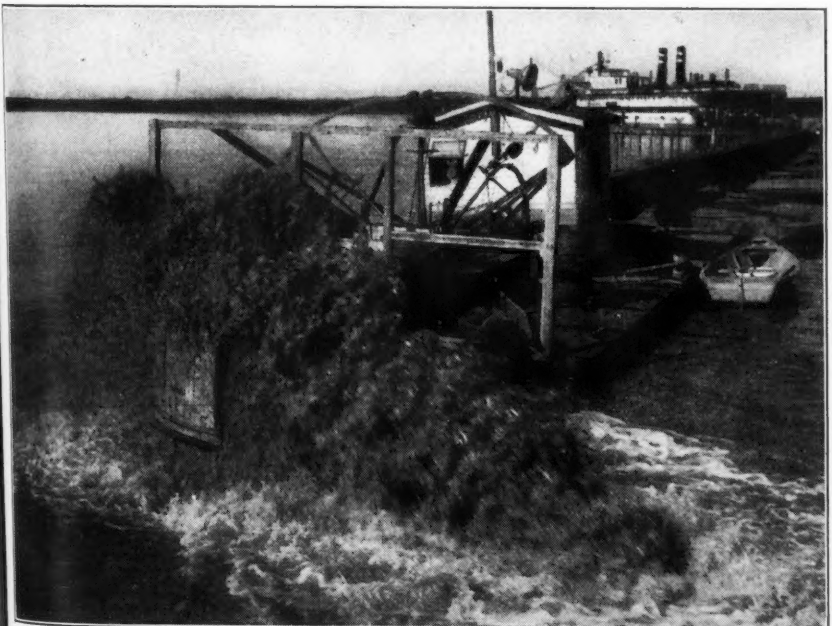


FIG. 11.—RELATION BETWEEN "BLOCKING" POINTS FOR TYPE 8 AND TYPE 12 MIXERS

In order to determine the most effective spacing in the pipe line, the two mixers, types 8 and 12, were installed at various distances from the end of the pipe line. Solids distribution samples were taken for several velocities and used as a means for determining the most effective spacing for satisfactory operation. Investigations on plain pipe had shown a concentration of material in the lower part of the pipe and a marked decrease in velocity within the area of maximum solid concentration. In determining the proper spacing for mixers, the location desired was the position in the line just upstream from the point at which concentration on the bottom would occur. A fairly uniform velocity distribution would occur in this location, thus utilizing all effects of the spiral motion. If the spacing of the mixers were too great, the material would all be concentrated on the bottom of the pipe and would require an amount of energy comparable to that originally expended to impart a spiral motion to the flow once more. The results of the spacing tests on the type 8 mixer were comparable with those shown in Fig. 7. As in the tests on the type 12 mixer presented by the author, a spacing of 20 diameters was indicated in order to obtain satisfactory mixing at the maximum distance between sections. Comparison of the spacing tests indicated that with a spacing of 20 diameters there was a better mixing of material through use of the type 8 mixer. However, the



(a) Plain Pipe; 13.3% Solids



(b) Type 12 Rifled Discharge Pipe; 18.1% Solids

FIG. 12.—DISCHARGE FROM A 32-IN. DREDGE; PUMP SPEED 150 RPM

mixing was not sufficiently increased to justify the $\frac{1}{8}$-diameter reduction in clear pipe diameter between types 8 and 12. Furthermore, in the light of the head-loss comparison discussed previously, it seems justifiable to state that type 12 is the better of the two designs.

The type 12 mixer, which the author states was superior to the other types tested during the first series, was selected for field tests. The dredge was located on the Mississippi River, north of Memphis, Tenn., in a selected reach of the river to insure the pumping of coarse materials. The results of these tests indicated that the type 12 mixer was superior to plain pipe. Fig. 12 shows the discharge from the end of the pipe for plain pipe and for rifled pipe. It is to be noted that the material is more thoroughly mixed when discharging from the rifled test line (Fig. 12(*b*)) as is evidenced by the decrease in spray at the end of the pipe. H. S. Gladfelter,[5] M. Am. Soc. C. E., has shown the distribution of solids across the test pipes for the conditions shown in Fig. 12. The sections presented by Mr. Gladfelter indicate, definitely, the mixing ability of rifled pipe. Results of these preliminary field tests led to the further development of a mixer more practicable for field installation. Additional mixers investigated are described in Table 1(*b*). The writer hopes that the author will enlarge on the results of the aforementioned field tests, thus advancing, another step, the knowledge of effect of rifling in pipe lines.

[5] *Transactions*, Am. Soc. C. E., Vol. 104 (1939), p. 1374.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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DISCUSSIONS

EFFECTIVE MOMENT OF INERTIA OF A RIVETED PLATE GIRDER

Discussion

By Messrs. J. M. Garrelts and I. E. Madsen, Francis P. Witmer, and A. W. Fischer

J. M. Garrelts,[30] Assoc. M. Am. Soc. C. E., and I. E. Madsen,[31] Jun. Am. Soc. C. E. (by letter).[31a]—There is some difference of opinion as to the behavior of the elements of a plate-girder beam under load. With the adoption of increased unit design stresses, it becomes important for engineers to examine some of the fundamental assumptions that have been used for designing such beams.

The authors' research represents an excellent attempt to learn more about the plate-girder beam by observing the action of test girders under load. The questions relating to the position of the neutral axis and to the moment of inertia of the beam are important and warrant investigation because they are uncertain quantities.

The writers are of the opinion that for a beam with open holes in both flanges (similar to series B in the tests) the neutral axis will remain at the center line for some range of loads, perhaps up to the working load. Since the stress-strain relation is taken to be the same in compression as in tension, the deformations would be equal in the two flanges as long as the action is in the elastic range of the material. Thus, until plastic deformation occurs on the entire flange cross section at an open hole, there is no reason for a shift of the neutral axis. The effect of stress concentrations at the edges of the holes would not affect this action appreciably.

When there are bolts or rivets in the holes, the deformations in the two flanges may also be essentially equal for some range of loads. In a region of pure moment, this action will depend on how well the rivets fill the holes, and

Note.—This paper by Scott B. Lilly, M. Am. Soc. C. E., and Samuel T. Carpenter, Assoc. M. Am. Soc. C. E., was published in October, 1939, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: December, 1939, by Messrs. William R. Osgood, Clyde T. Morris, B. R. Leffler, E. Neil W. Lane, Lewis E. Moore, W. E. Black, and L. E. Grinter; January, 1940, by Messrs. Charles M. Spofford, E. Mirabelli, Edward Godfrey, Walter H. Weiskopf, and C. D. Williams; and February, 1940, by Messrs. Alvin B. Auerbach, Jonathan Jones, and R. L. Moore.

[30] Associate Prof., Civ. Eng., Columbia Univ., New York, N. Y.

[31] Asst. Research Engr., Fritz Eng. Laboratory, Lehigh Univ., Bethlehem, Pa.

[31a] Received by the Secretary January 18, 1940.

on the friction forces developed by the rivet heads. If the rivets or bolts fill the holes sufficiently well to offer resistance to deformation of the flange materials, the transmission of forces across the net section will be of a different nature in the two flanges and will produce an unsymmetrical effect. The friction forces acting on the rivet heads will generally develop some tensile or compressive stresses in the rivet heads, thus providing some splicing action at that section.

The writers are in doubt as to what is meant by the statement (see heading "Conclusions"): "It is believed that the data from these tests point to the acceptability of the gross moment of inertia for design purposes." Do the authors refer to design computations for deflection, or for stress? If the latter is meant, the writers do not agree. A consideration of statics will show that the average stress on a cross section through a rivet hole must be larger than that on a gross cross section. In addition to the expected variation of stress due to stress concentration adjacent to the holes, strains measured on 0.5-in. gage lengths by the writers show a higher average unit strain at a cross section through a rivet hole than at a cross section taken between rivet holes. The 10-in. Whittemore strain gage, used in the authors' tests, measures the over-all average strain for all cross sections included in the 10-in. length, but does not give the average strain at the minimum cross section.

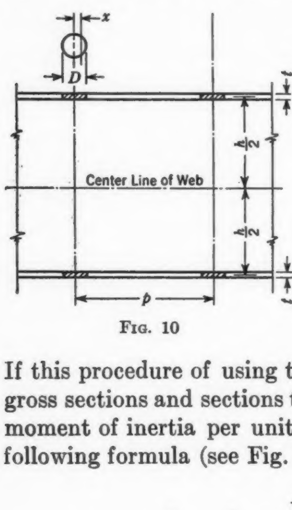

FIG. 10

It seems inconsistent to use the gross moment of inertia for proportioning the parts of a girder according to allowable stresses and an "effective" moment of inertia for computing deflections. The writers are of the opinion that an "effective" moment of inertia is an unnecessary refinement, because it represents the action that occurs only for the case of open holes, or for that in which the bolts or rivets do not fill the holes and therefore offer no resistance to deformation of the flange materials. If this procedure of using the weighted average of the moments of inertia for gross sections and sections through the holes is to be used, however, an average moment of inertia per unit of length of the girder can be determined by the following formula (see Fig. 10):

$$I_a = I_G - \frac{1}{p}\int_{-D/2}^{D/2}\left(2\sqrt{\frac{D^2}{4} - x^2}\right) t\, N \left(\frac{h}{2}\right)^2 dx$$

$$= I_G - (N\, t\, D)\left(\frac{h}{2}\right)^2 \frac{\pi}{4} \times \frac{D}{p} = I_G - I_H \times \frac{\pi}{4} \times \frac{D}{p} \quad \text{......(9)}$$

in which I_a = average moment of inertia per unit of length; N = number of rivet holes at cross section; D = diameter of rivet holes; p = rivet pitch; and I_H = the moment of inertia of the area of the holes, taken at the diameter of the holes.

Assuming

$$m = \frac{I_a}{I_G} = 1 - \frac{I_H}{I_G} \times \frac{D}{p} \times \frac{\pi}{4} \quad \text{.................} (10)$$

values of m are shown in Table 9.

TABLE 9.—COMPARISON OF VALUES OF $m = \frac{I_a}{I_G}$

Section	Value of m by experiment	Computed value of m	Percentage deviation	Value of m by experiment	Computed value of m	Percentage deviation
C1	0.970	0.971	+0.1	0.928	0.942	+1.5
C2	0.970	0.973	+0.3	0.936	0.945	+1.0
C3	0.962	0.975	+1.3	0.916	0.951	+3.8
C4	....	....	....	0.930	0.951	+2.3
D3	0.980	0.975	−0.5	....	....	....

The computed values of m check fairly well with the experimental values. However, the experimental values are not a true representation of $\frac{I_E}{I_G}$. This is readily seen when the method for determining m is considered. The value of m was computed as the ratio of the loads required to produce equal deflections for a given girder section and the corresponding section of series A. Since there were rivet holes in the vertical legs of the flange angles of the series A sections, the experimental values for m represent a comparison of the effective moment of inertia (I) for series B, C, and D with the effective I of series A.

The value of the factor k in the authors' equations is $\frac{\pi}{4}$, the same as the coefficient in Equations (9) and (10) for I_a. This quantity is a shape factor and depends only on the shape of the holes.

The exact behavior of the elements in a plate girder is unpredictable because of the uncertainty of the action of the rivets; but tests such as those that the authors report will aid in a correlation of the analysis with the possible structural action for which the girder should be designed.

FRANCIS P. WITMER,[32] M. AM. SOC. C. E. (by letter).[32a]—The interesting experiment reported in this paper has verified, satisfactorily, the general understanding among structural engineers that deflections of a plate girder are in practical accord with the gross, rather than with the net, moment of inertia. A similar relation, however, cannot be accepted so readily as regards maximum effective unit stress in the tensile flange. In the investigation of old plate girder structures whose sufficiency for modern loading was in question, the writer has found it convenient to use the method described below for the determination of these tensile unit stresses. Assume the girder to deflect in accordance with its gross moment of inertia. Since the section is symmetrical, this places the neutral axis at the center of the web plate, and also presupposes that the rivets hold the parts together with sufficient force to prevent slip and, therefore, to develop stresses which are in accord with bending theory.

[32] Director, Civ. Eng., Univ. of Pennsylvania, Philadelphia, Pa.

[32a] Received by the Secretary January 30, 1940.

If c is the distance from the neutral axis to the extreme fiber, the maximum unit stress on the gross section will be $s = \frac{M\,c}{I}$. If the distance from the neutral axis to any flange rivet line (either through the web or through cover plates) is y, the unit stress on the gross section at this line of rivets will be $s' = s\frac{y}{c}$.

This unit stress will occur in the part of the flange between rivets. A rivet hole will cut from the flange section an area equal to $2\,D\,t$, in which D is the diameter of the hole and t is the thickness of flange angle, or, for cover-plate rivets, the combined thickness of angle and cover plates for two lines of rivets.

The rivet hole may now be regarded as cutting from the section an area whose total stress equals $s'\,(2\,D\,t)$ and causing this stress to be detoured around the rivet and ultimately divided up over the entire net section of the flange. If A represents this net section, a unit stress of $\frac{s'\,(2\,D\,t)}{A}$ will be added to each unit stress in the gross flange section between rivets. Adding this increment to the maximum extreme fiber stress, s, in the gross section, will give the maximum tensile unit stress in the section.

This method of procedure is inconsistent in that it assumes the gripping action of the rivets to be sufficient to cause a distribution of fiber stresses between rivets which is proportional to the distance from the neutral axis; but it does not consider the frictional force under the rivet head as sufficient to prevent the increment of stress, due to the flange material cut out by the rivet hole, from being carried around the rivet. The results, however, are believed to be more nearly correct than those by either the method of net moment of inertia or that of net flange area. For a girder without cover plates, the result will usually be nearly the same as by the net flange area method; but if there are cover plates, it will generally be somewhat less than this value.

A. W. Fischer,[33] Esq. (by letter).[33a]—The experiments and results reported in this paper were conducted with great care and refinement.

According to all the tests, the neutral axis remained fairly close in position to the center of gravity of the gross section; and this seems reasonable because the rivet holes in the tension flange do not remove much metal as compared to the entire volume of the tension flange. With a 5-in. pitch there is slightly less shift than at a 2.5-in. pitch, and that is as it should be; and, for average conditions, the writer will agree that, for the deflection of a riveted plate girder, it will be satisfactory to assume the neutral axis at the center of gravity of the gross section, and to use the gross moment of inertia. The computed deflection will then be a trifle less than it really is by experiment; but the variation will be small, and may never be more than 3% in error.

A riveted plate girder acts similarly to a truss. Every engineer will deduct the rivet holes in the tension flange of a steel truss, and for that reason it seems that rivet holes should be deducted from the tension flange of a riveted plate girder to ascertain the true stresses.

[33] Asst. Structural Engr., Construction Service, Veterans Administration, Washington, D. C.

[33a] Received by the Secretary February 5, 1940.

The authors failed to secure a true measure of the stress in the net section, for the tension flange, because of their testing procedure. The true stress at the section where the rivets occur cannot be computed from the readings of a 10-in. gage length. Only an average stress can be computed; and this average will be less than the actual at the rivet section. Furthermore, it is impossible to determine the true tensile stress at the section of a line of rivets if round holes are used. The only solution is to use slotted holes at the section where the stretch is to be measured, the slotted part being, say, 1 in. long, and the width being the diameter of the rivet hole. If the slotted part is actually 1 in. long, if a gage length of 1 in. is used, and if the average of several positions across the net area of the tension flange is used, then from such a test the average of the calculated results will represent the true stress at that section as closely as an experiment can be made to determine it. Tests should first be performed using slotted holes at the net section tested, and then the load should be removed. After the load is removed rivets shall be placed in the slotted holes and tested again to ascertain if the grip of rivet heads change the stresses.

The plate girder tested by the authors was rather shallow; a representative girder of reasonable depth should be tested, and several cover plates should be used so that the effect of rivet-head grip on the outer cover plate and the outstanding leg of the angles at the net section will be reduced to an average minimum.

One item that is not clear to the writer is just how the 10-in. gage length was converted to the true stretch of the bottom tension flange. A slight curvature in the beam seems to indicate that the actual gage length measured was the chord length and not the true arc length. It seems that the shorter the gage length the more definitely the true stretch can be ascertained.

The I_E-value as given in Equation (5) is too large. Before it is adopted, additional tests should be performed on representative girders, reasonably deep, and including several cover plates. The writer will agree that the pitch of the rivets in the tension flange should be used in the general equation so as to take care of the slight shift of the neutral axis, and the writer would not use a greater value for I_E than the following for the design of a riveted plate girder:

$$I_E = \frac{p\, I_N}{p + \left(\frac{I_G}{I_N}\right)^4 - 1} \quad \ldots\ldots\ldots\ldots\ldots\ldots (11)$$

in which I_N is equal to I_G minus the moment of inertia of the rivet holes in the tension flange only, about the gravity axis of the gross section. The section modulus then equals I_E divided by the distance from the gravity axis of the gross section to the extreme tension fiber.

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DISCUSSIONS

TRANSIENT FLOOD PEAKS

Discussion

BY MESSRS. GORDON R. WILLIAMS, AND DONALD M. BAKER

GORDON R. WILLIAMS,[14] ASSOC. M. AM. SOC. C. E. (by letter).[14a]—The phenomenon with which this paper deals is a special type of flood wave which, in the regions under consideration, is also in part a debris wave. A serious deficiency in the paper is that the author has failed to separate the characteristics of the wave from those of the flow of water which follows the wave. By tabulating the hydraulic properties of the wave fronts, the implication is made that these values can be substituted in a formula for normal open-channel flow (the Manning formula) and that the results will indicate that phenomenal rates of water discharge, entirely inconsistent with the recorded rates of rainfall, have occurred. The purpose of this discussion is to reveal, in the light of present knowledge of flood waves, the inadvisability of such a procedure.

A flood wave with a turbulent, vertical front has been designated by different terms by various writers on the subject. Harold A. Thomas,[15] M. Am. Soc. C. E., uses the term "hydraulic bore"; Hunter Rouse,[16] Assoc. M. Am. Soc. C. E., the term "gravity shock wave"; and A. Schoklitsch[17] the term "flood surge." The term bore will be used in this discussion.

For a bore to be formed it is necessary that there be a sudden concentration of water in a channel and that conditions in the channel be such that the concentration of water will produce a depth which is more than twice the initial depth of flow. This condition is shown diagrammatically in Fig. 8, in which d_2, the maximum depth of the wave front, is more than twice d_1, the initial depth of flow. If $d_2 < 2\,d_1$, the wave front assumes a normal wave form, and no bore is formed.

NOTE.—This paper by Henry B. Lynch, M. Am. Soc. C. E., was published in November, 1939, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: January, 1940, by Ivan E. Houk, M. Am. Soc. C. E.

[14] Associate Hydr. Engr., U. S. Engr. Office, Baltimore, Md.

[14a] Received by the Secretary January 10, 1940.

[15] "The Hydraulics of Flood Movements in Rivers," by Harold A. Thomas, *Engineering Bulletin*, Carnegie Inst. of Tech., 1934, pp. 24, 28–29.

[16] "Fluid Mechanics for Hydraulic Engineers," by Hunter Rouse, *Engineering Societies Monographs*, McGraw-Hill Book Co., Inc., New York, N. Y., 1938, pp. 382–388.

[17] "Hydraulic Structures," by Armin Schoklitsch, published by A.S.M.E., 1937, pp. 121–128.

The author expresses the theory that a bore is formed when there is an abrupt increase in rainfall rates following a period of prolonged rainfall. This theory can be substantiated in part because, if there has been rain for some time, high initial infiltration rates have been succeeded by lower rates, much depres-

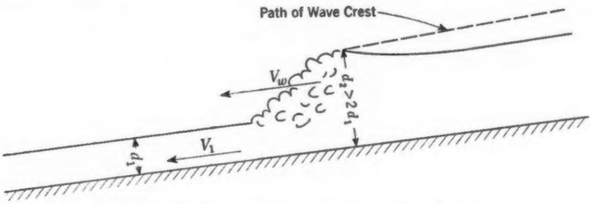

FIG. 8.—PROFILE OF A DEBRIS-FREE BORE

sion storage has been taken up, and conditions are "ripe" for quick concentration in the stream channels of the increased rainfall. If a stream flows through a narrow canyon, where there is little valley storage, the concentration of water will produce a depth which is large in relation to the depth of the initial flow, and a bore will be formed. Thus it is evident that there must be both a meteorological and a hydraulic cause for a bore and that the increased rainfall is only the meteorological cause.

Conditions in the burned areas mentioned in the paper were probably more conducive to rapid concentration of water in the upstream channels than in the unburned areas, but, as will be indicated subsequently herein, such conditions may have had a more marked effect on the form of the flood wave and on the debris movement than on peak rate of water discharge.

The discussion thus far has considered a bore which is composed entirely of water. The bores that occur in the western mountains pick up tremendous quantities of debris and carry them along in suspension or as bed load. Where there are steep bottom slopes it is not impossible that debris may occupy 50% or more of the cross-sectional area of the wave front.

Where the debris content is extraordinarily large, the solids may act as a moving dam, the effect of which would account for some of the large cross-sectional area below the floodmarks. Any debris left behind by the initial wave would be carried on by the less heavily burdened water which followed the wave, and thus remove the last evidence of the true water area at the maximum stage. Evidently the movement of debris was greater from the burned areas than from the unburned areas. In extreme cases the conditions were probably not unlike the "detritus avalanches" of mountainous European streams discussed by Otto Franzius.[18]

Statements in regard to the consistency of the fluid mass are contained in the last sections of the paper but should have been given more prominence in the first part, especially when the channel areas below the floodmarks are considered. Descriptions of the debris waves in La Cañada Valley, California,

[18] "Waterway Engineering," by Otto Franzius (translated by Lorenz G. Straub, Assoc. M. Am. Soc. C. E.), *The Technology Press*, Mass. Inst. of Tech., Cambridge, Mass., 1936, p. 15.

flood and theoretical discussions of their characteristics have been presented by Harold C. Troxell, Assoc. M. Am. Soc. C. E., and John Q. Peterson.[19]

For the purpose of further discussion, let it be assumed that the cross-sectional area of the wave front is composed entirely of water. Under conditions which exist in mountain canyons it is entirely probable that the depth of the bore is greater than the depth of the less turbulent water which is flowing after it, as indicated in Fig. 8. Some of this greater depth is no doubt due to entrained air. The profile defined by the floodmarks would be as shown in the dotted line in Fig. 8. If the slope of such a profile is used in the Manning formula to determine the mean velocity of the water particles following the wave, and if the area under the profile is used as the area of flow for these water particles, an obviously incorrect result will be obtained. The velocity of the wave front is a velocity of wave propagation, and, except in the case of a dry channel, is always greater than the velocity of the water particles that follow. The velocity of wave propagation cannot be obtained by the Manning formula and, if it could be so obtained, it could not be applied to the cross-sectional area of the wave to determine the discharge in the channel back of the wave. Mr. Schoklitsch has derived a formula for the velocity of the bore, V_w, which may be written in the form

$$V_w = V_1 + \sqrt{g\left[d_1 + \tfrac{3}{2}(d_2 - d_1)\right]} \qquad (2)$$

The water in a bore is in a state of extreme turbulence, largely in the form of a roller with a horizontal axis. Water is supplied to the bore both from the water that is being overtaken and from the following flow.

For the following reasons, the channel areas, wetted perimeters, hydraulic radii, and surface slopes given in this paper cannot be used to determine peak discharges when the form of the flood wave is a bore:

(*a*) The area during the passage of the wave front probably contained a large percentage of solids in transit and in some cases there may have been a damming effect;

(*b*) The water in the maximum area was in a state of extreme turbulence, characteristic of a vertical wave front, and the average velocity of the water particles relative to the stream bed was not that of the wave front;

(*c*) The area was larger than the area occupied by the less turbulent water which followed the wave; and

(*d*) The surface slope obtained from the floodmarks was defined by the crest of the wave and may not have been indicative of the slope of the water back of the wave.

The writer realizes that this discussion offers no solution for obtaining the discharge following a bore. The nearest approach would seem to be in measuring short time changes in storage in debris basins and reservoirs at the time of a flood. The rate of inflow of water could be obtained by subtracting from the increase in storage the increase in the quantity of solids that were stored. The difficulty would lie in the fact that it would be necessary either to assume that

[19] "Flood in La Cañada Valley, California, January 1, 1934," by Harold C. Troxell and John Q. Peterson, U. S. Geological Survey *Water-Supply Paper No. 796–C*, 1937.

all the increase in the volume of solids came at the time the peak entered the storage basin or to assume that the increase in solids was proportional to the changes in total storage; and neither assumption would be strictly correct.

There are several places in the paper where the use of terms or descriptive phrases is not in accord with accepted usage in the science of hydrology. In Fig. 1 the depth of flow preceding the "surge" is designated as "depth of runoff." This latter term is always applied to the volume of water which an area yields in a storm, or longer period of time, expressed as a uniform depth in inches over the entire area. Use of the term "depth of runoff" in Fig. 1 is misleading.

Again, under the heading "Flood of January, 1934," there is reference to soil having attained a "high degree of saturation." Except in a special chemical sense, it is not possible to have a "degree of saturation." Furthermore, the use of the word "saturated" is not advisable in reference to soil cover as experiments have indicated that infiltration continues at a reduced and steady rate even after a period of prolonged rainfall. However, it is conceded that, if the soil forms only a thin layer over rock, infiltration may become negligible.

DONALD M. BAKER,[20] M. AM. SOC. C. E. (by letter).[20a]—This paper adds some interesting and valuable information to the small amount now available in connection with the phenomenon designated by Nathan C. Grover, M. Am. Soc. C. E.,[21] as the "standing flood wave," and points to the following conclusions:

1. Protection against flood flows may be entirely inadequate under certain combinations of physical and meteorological conditions. This applies particularly to floods from small, steep, drainage areas, which depend solely upon improved channels, the capacities of which have been developed by the usual rational method (which involves such factors as intensity of precipitation, time of concentration of flow, percentage of rainfall appearing as runoff, etc.).
2. The waves or surges described in the paper, although of great magnitude and destructive force, are of relatively small volume or cubical content.
3. Denudation of the surface of the drainage area of vegetal cover by fire or other means, although not the sole cause of the formation of such waves, apparently results in a great increase in their magnitude and destructive force.
4. In view of the magnitude of such waves, it would not appear that the construction of check dams in the main channels—even if they were of a permanent nature—would serve any useful purpose as a protective measure against their occurrence or formation; nor would such a program involve complete coverage of the entire drainage area. If the cost of such construction could be kept within reasonable limits, construction of a permanent type of check dam in the smaller channels over a portion of the drainage area might serve to increase time of concentration of the water flowing from such checked area, and so reduce peaks, although it is questionable if such measures would be as economical as construction of debris basins.

[20] Cons. Engr., Los Angeles, Calif.

[20a] Received by the Secretary January 29, 1940.

[21] "Standing Waves in Rivers," by Nathan C. Grover, *Transactions*, Am. Soc. C. E., Vol. LXXXV (1922), p. 1400.

The combination of physical and meteorological conditions which produced the waves described in the paper, in the canyons draining the area burned in 1933, included:

(*a*) An abrupt increase in rainfall intensity over a wide area, amounting to nearly 1.0 in. per hr, or 260% of the preceding intensity rate;

(*b*) A state of almost complete saturation of the surface of the drainage area at the time of the occurrence of the increase in rainfall intensity, resulting in a runoff coefficient approaching 100%;

(*c*) An almost complete absence of brush and litter on the surface of the drainage area, except in the beds of the channels, this condition affording opportunity for high velocity of surface runoff;

(*d*) Channels located in easily eroded material, and containing a considerable amount of debris accumulated over a long period; and

(*e*) Steep surface slopes on the drainage area, ranging from 25% to 50%, or more, and high channel gradients throughout, the gradients being somewhat less in the lower reaches of the canyons.

If it is assumed that the runoff on the slopes and in the channels follows the Manning formula, it can be shown that, with conditions of constant slope and roughness, where depth of flowing water approximates the hydraulic radius, such depth varies with the 0.6 power of the quantity, and velocity with the 0.3 power of the quantity.

With a 100% runoff factor, an increased intensity equal to 260% of the preceding intensity will increase the rate of runoff in the same proportion, and under the foregoing assumptions depth would be increased 178% and velocity of flow 133%. Furthermore, since slopes and channel gradients in the higher elevations are greater than at lower levels (probably upwards of 50%), the time of concentration of water in the upper reaches of the drainage area would be shorter and runoff from increased rainfall would accumulate in the channels more quickly at higher elevations. The fan-like shape of the heads of the canyons would also tend to increase this rate of accumulation.

Were all of the channels on the drainage area straight and smooth, offering little frictional resistance, the increased flow would have a tendency to "slide" smoothly along the channels, with a gradual "welling up" of the flow. However, the channels are crooked, rough, and filled with boulders and trees. These conditions result in extremely turbulent flow, particularly in the first increments of increased flow, the energy of the water being dissipated and its velocity reduced. Furthermore, some of the first portions of the increased flow are accumulated as channel storage; and the rising stream picks up debris, particularly brush and trees, which tend to pile up and retard the flow. The general result of these conditions is that the first increments of increased flow do not move downstream as fast as does the later discharge. This flow, especially the portion at the surface and near the center of the stream, moves faster than the water on the sides and bed, and soon a small wave is created which is very turbulent in character and filled with debris. The faster moving water upstream spills over this wave, its velocity being dissipated.

The cumulative effect of these conditions is the formation of small standing waves in each of the small channels, which concentrate into larger waves and

ultimately into a large standing wave in the main channel. This large standing wave acts as a barrier or moving "dam" as it advances downstream at a velocity less than that of the water behind it. This onflowing water keeps piling up higher and higher behind the "dam" thus formed by the standing wave.

Because of its turbulence, the water along the wave front has much greater erosive and transporting power than has the water flowing behind it, and is able to pick up and move large loads of debris. Large boulders such as are described in the paper are set in motion and literally "bounced" down the channels.

The fact that this advancing wave carries much material considerably larger than sand throughout its entire cross section is evidenced by the existence of small rock chips, $\frac{1}{8}$ in. or more in diameter, firmly embedded flush with the surface of a hard-pine telephone pole to heights of 15 ft or more above the ground, the pole being located on the north side of Foothill Boulevard just west of the Pickens Canyon crossing.

Three considerations lead to the conclusion that the volume of water in these standing flood waves was not great, and that their duration was very short. The cross section of the flooded channel in Haines Canyon at the U. S. Geological Survey gaging station, as given by the author, had an area of 299 sq ft, with a depth of water between 11 and 12 ft. The capacity of the Haines Canyon debris basin between the elevation where the water stood prior to the occurrence of the wave and the maximum depth of overflow of about 2 ft on the spillway crest is apparently between 15 and 17 acre-ft, or from 650,000 to 740,000 cu ft. A depth of 2 ft over the spillway crest would represent a flow of the order of 600 cu ft per sec. A discharge of 5,000 cu ft per sec in excess of this quantity would have filled the basin in a little more than 2 min, and one of 10,000 cu ft in a little more than 1 min.

If Manning's formula is used, and the hydraulic properties of the channels and cross sections as given in Table 1 are taken, velocities ranging from 20 to 70 ft per sec are developed. Such velocities would have far greater erosive effect than is evidenced on the ground or in photographs taken immediately after the flood. It is probable that the slope of the water surface upstream from the front of the wave was considerably less than the natural slope of the channel. This would cause the wave to take the shape of a wedge, with a base equal in area to the cross section of the channel, and a height equal to the distance between the wave front and the point upstream where the surface slope of the flowing water was equal to that of the natural channel.

At the location where Pickens Canyon crossed Foothill Boulevard, a concrete box culvert about 11 ft by 18 ft in cross section carried the wash under the paved highway, with about 4 to 5 ft of earth fill over the top of the culvert. Apparently water reached a height of 15 ft or more over the paved section of the highway. Even with velocities of 15 to 20 ft per sec, a flow of any duration undoubtedly would have washed out the highway embankment and caused great erosion downstream from it; but such was not the case.

The ratios of cross-section areas of channels draining lands burned in 1933 and 1927 (with partial recovery of brush cover), and unburned areas was 442, 166, and 100%, respectively. The effect of fire upon the magnitude of the flood waves, due to denuding the surface of the drainage area, is thus indicated

forcibly. Although a stand of brush recovers fairly rapidly in Southern California, it requires a much longer time to accumulate litter, which has great retarding effect upon runoff.

With depths of cross sections in the channels draining the area burned in 1933 (as indicated in the author's notes) ranging from 10 to more than 25 ft, it is questionable whether check dams in the main channel, even of a very permanent nature, would have had any effect in reducing the magnitude of these flood waves. They might even have increased such magnitude by their retarding effect upon the advancing wave front. If such dams were placed in all of the side channels, and spaced so as to reduce the gradient therein by 50%, the velocity would have been reduced only 30%; but the velocity of surface runoff would not have been retarded. Therefore, the increased flow would have been ponded or stored in the smaller channels for a longer period of time but in all probability would have reached the main channels in flows of the same magnitude as it would have without such check dams.

Construction of check dams on side channels on, say, one half of the drainage area might have the effect of retarding the time of concentration from this part of the area, and thereby reduce main channel peaks by prolonging the outflow. Necessarily, such a program would require very careful planning and, in all probability, would be considerably more expensive than the present method of constructing debris basins below the mouths of the canyons and above the protected channels.

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DISCUSSIONS

WATER SUPPLY ON UPPER SALT RIVER, ARIZONA

Discussion

BY DANA M. WOOD, M. AM. SOC. C. E.

DANA M. WOOD,[8] M. AM. SOC. C. E. (by letter).[8a]—Long-range predictions of rainfall and runoff have been attempted by many investigators, but so far with few practicable results. This does not mean that attempts should be discouraged. Sooner or later someone may find a method of sorting the many complicated actions and reactions of atmospheric conditions with their causes and effects and develop at least a hazy picture of what may happen during the next few years. At present, one can scarcely claim that predictions for the next few days or weeks are reliable.

The need for such predictions in power estimating is not great. (This does not apply to needed predictions for a few days for operating a power system.) Occasionally, it would be helpful to know whether the next year or two might be of wet, dry, or average type, in order to provide protection for some inadequacy during a development or construction period. However, careful analysis of sufficiently long and reliable runoff records, their proper interpretation, adequate and proper design of the power scheme, and the development of suitable operating guides and rules for obtaining the maximum efficiency for the purposes to be served are to be preferred to wild guessing as to future runoff. Good power planning takes into account the sequence of good and bad runoff years in the past on the theory that history may repeat itself. Provision should be made for the variations that surely will come by applying suitable contingency factors to the records. The length and general accuracy of the record determine to what extent it should be discounted. Storage, pondage, steam relay, and interconnections are all factors in the solution of the problem of insuring against the variations in nature.

Although careful consideration of Figs. 2 and 3 clears up any question in the reader's mind, it would nevertheless be helpful if they were designated as

NOTE.—This paper by John Girand, Assoc. M. Am. Soc. C. E., was published in December, 1939, *Proceedings*. This discussion is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion of the paper.

[8] Prin. Hydr. Engr., TVA, Knoxville, Tenn.

[8a] Received by the Secretary February 2, 1940.

curves of annual total runoff to differentiate from the more usual monthly, weekly, or daily hydrographs and duration curves.

Table 1 is of doubtful value to any one. It is not correct mathematically; also, it assumes the same relative distribution during the year, regardless of its type. That this is far from what actually happens scarcely needs proof. This one assumption can lead to more error in estimating the potential power than even poor analysis and interpretation of actual records. The use of monthly estimates of runoff will ordinarily result in power estimates that are at least 10% too high, unless available storage is so large as to compensate for variations in flow during the month.

Many who have studied the problem believe that rainfall and runoff occur in cycles, varying both in amplitude and magnitude, just as, on the ocean, variations from the big wave to the little ripple depend upon whether the forces creating them are large or small, or whether they are acting mostly in one direction or opposing one another.

For purposes of making reliable power estimates, the writer advocates the analysis of runoff records, rather than rainfall records, because of the many factors affecting rainfall-runoff relations. Fig. 7 indicates the cycles that have occurred in the runoff of the Tennessee River at Florence, Ala. Note their irregularity. Such a study is prepared by plotting the percentage of duration of each significant rate of flow in each year on a horizontal line representing that year, and then connecting the points of equal flow through all years. Such a drawing might be termed a contour map of runoff. Where the lines reach nearest to the 100% of the time axis, the runoff is high; where they reach nearest the 0% axis, the runoff is low.

The author mentions the use of a helpful tool—the ratio to the mean flow. So far as the writer knows, he was the first to use this method of analysis.[9] Subsequently, the Run-off Committee of the Boston Society of Engineers[10] made use of it. Tables (and curves) can be prepared from duration-of-flow studies by computing the ratio of the flow in cubic feet per second (or cubic feet per second per square mile) to the average flow for the entire period under analysis, for significant percentages of the time. The result may be called relative distribution tables or curves. Gaging records showing no similarity when plotted on the usual duration-curve basis will show close similarity in distribution of runoff or will show differences that will class the runoff as good or bad for the desired purpose of development—say, power. Comparisons should be made for the same period of time. The point of 100% availability has little significance whenever regulation affects it to a marked extent.

The method can be used for extending a short record to a longer one by comparison with a gaging station showing similar distribution of runoff in the shorter period. Sites far apart geographically can be compared. The method permits comparisons of runoff between small and large drainage areas and between basins having low and high unit runoff. It can be used, in an extreme case, for determining a possible duration curve by assuming the percentage runoff

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[9] "Comparisons of New England Run-Off Data," by Dana M. Wood, Stone & Webster *Journal*, Boston, Mass., February, 1917.

[10] "Report of the Committee on Run-Off," *Journal*, Boston Soc. of Civ. Engrs., Vol. 9, October, 1922, p. 171, and Figs. 2 and 3.

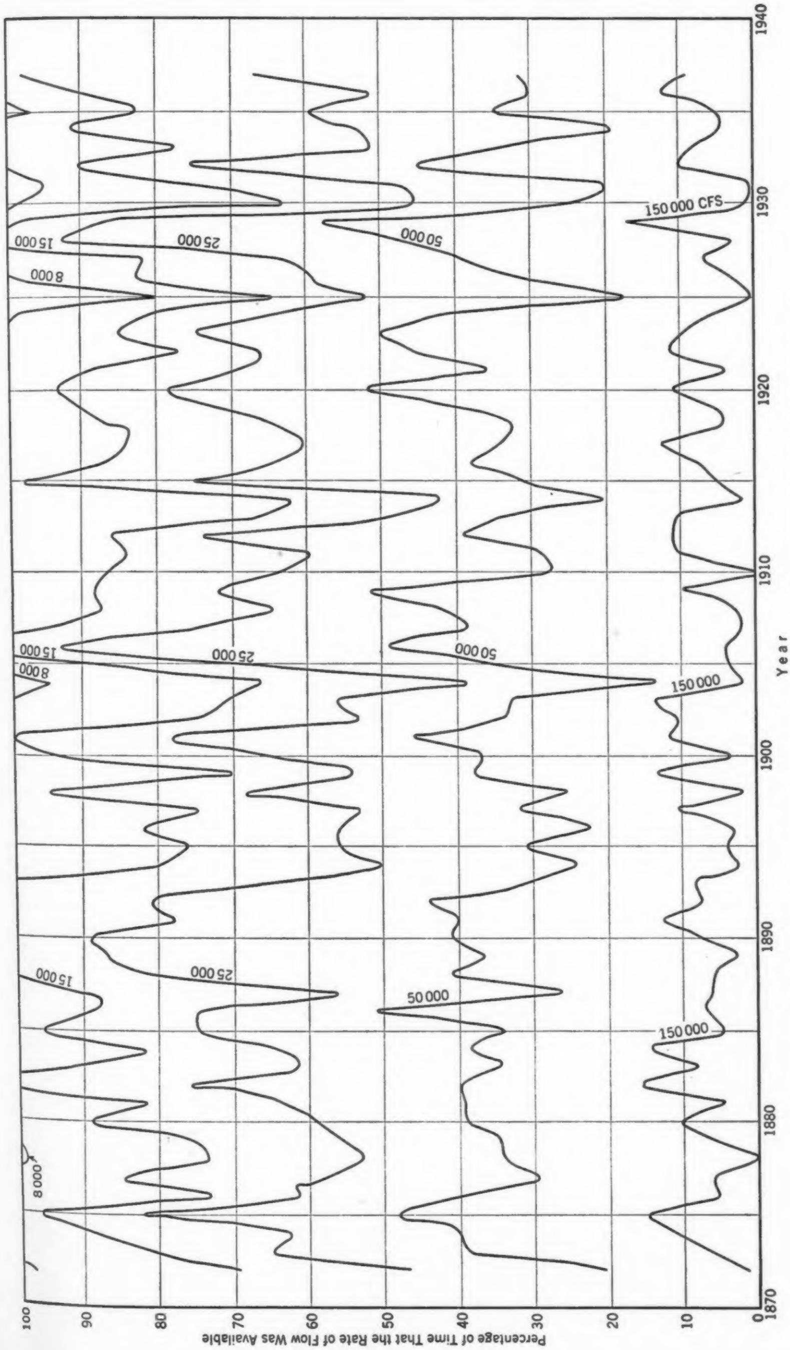

FIG. 7.—STUDY OF RUNOFF CYCLES BY CALENDAR YEARS, TENNESSEE RIVER AT FLORENCE, ALA.

from rainfall records where no runoff records exist, and by assuming its distribution as at some actual gaging station, provided that good judgment is used in choosing the runoff record that is assumed typical of the site under consideration. As an extreme example of the latter, the writer once estimated the runoff from a watershed in Japan by using a gaging record in the Rocky Mountains in the United States, with results that checked surprizingly well with subsequent current-meter records. It has several other useful applications, supplementing other better-known methods of analysis.[11]

[11] Discussion by Dana M. Wood of "Rainfall and Run-Off Studies," by the late C. E. Grunsky, Past-President, Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. LXXXV (1922), p. 106.

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DISCUSSIONS

CHANNELIZATION OF MOTOR TRAFFIC

Discussion

BY MESSRS. W. L. WATERS, C. J. TILDEN, AND T. M. MATSON

W. L. WATERS,[7] M. AM. SOC. C. E. (by letter).[7a]—The channelization of vehicle traffic by separating islands at street crossings has probably been practiced for 100 years in the large cities of Europe; and, although Mr. Kelcey's paper is somewhat academic, it is most valuable as a collection of up-to-date ideas on the subject. As is frequently the case in such discussions, however, he confines it almost entirely to motor vehicles, and the unfortunate pedestrian receives only one half page out of a total of 25 pages. As the writer walks probably 3,000 miles each year, and as the spectator often sees more of the game than the players, some of his comments may be of interest.

Motorists on any particular street or highway can be divided into those who are familiar with the road, and those who are not. The separating islands at crossings, as sketched in the paper, would usually be a help to those familiar with the road or crossing. They would facilitate the channelization of the traffic. For those unfamiliar with the road, however, the reverse would often be the case. As Mr. Kelcey states, a motorist should keep his head up, watch the road 200 or 300 ft ahead, and plan accordingly; but if the traffic is so light that he can see the islands 200 or 300 ft ahead, the islands are scarcely necessary. It is when the traffic is dense and moving fast that they are required; and in that case the motorist, because of the vehicles, cannot see the islands until he is "right on top of them." If he is unfamiliar with the road, he then is likely to become confused and end by stalling the traffic. The complicated channelization approaches to some modern bridges are an example of plans which, while seeming to be perfect on the drawing board, are only too frequently a "headache" in practice.

In 1938 a party of English schoolboys made a motor tour through the United States and Canada. Subsequently interviewed by the press, one of them said that what interested him most was the unhesitating way in which motorists in this country do what they want to do. This was a very gracious

NOTE.—This paper by Guy Kelcey, M. Am. Soc. C. E., was published in December, 1939, *Proceedings*. This discussion is published in *Proceedings* in order that the views expressed may be brought before all members for further discussion of the paper.

[7] Cons. Engr. (Bury & Waters), New York, N. Y.

[7a] Received by the Secretary January 2, 1940.

way of expressing his astonishment at the antics of the American "road hog." It is the "I want what I want when I want it" spirit, which is a national characteristic, from which no one in America is exempt. When a person is at the wheel, he thinks pedestrians should be abolished and that most of the other motorists are "road hogs"; whereas, when he walks, he thinks that the average motor car is a public nuisance.

There are two practical ways of handling traffic. One is by strict regimentation and the other is by mutual "give and take"—that is, by consideration of the rights of others. In Europe, regimentation is the rule except in England and the Scandinavian countries, where public spirit seems to be effective. With its "rugged individualism" the public in the United States refuses to be regimented. Therefore, the "road hog" must be taught to behave like a gentleman.

With the battleship type of construction, motorcars are becoming like army tanks, and the fatalities to motorists are decreasing. On the other hand, pedestrian fatalities are increasing rather than decreasing. "Shotgun" crossings were threatened in St. Louis, Mo., in 1905, when limited stops for trolley cars were first introduced to speed up the traffic. Those who have seen the unpleasant atmosphere that sometimes develops when accidents occur to pedestrians or children may think that, if such episodes are to be avoided, more public spirit must be developed both in the motorist and in the pedestrian.

The extent to which channelization islands at crossings will improve conditions is uncertain. As Mr. Kelcey states, when speeds increase the accidents increase, and the introduction of islands indiscriminately at crossings may have the effect of increasing speeds. The rush of pedestrians to an expected island refuge, only to find it already crowded, may make conditions worse than if there were no islands; and many pedestrians dislike incurring the risk of being isolated on an island for perhaps several minutes. It is evident, therefore, that such channelization of traffic can in no way be a substitute for "stop-and-go" traffic lights.

The two weak points in the theory of channelization, as so ably explained by Mr. Kelcey, are (1) that it assumes all motorcars are familiar with the arrangements at each crossing, and (2) that it largely ignores the pedestrian.

C. J. TILDEN,[8] M. AM. SOC. C. E. (by letter).[8a]—The fundamental principles of keeping highway traffic in line are presented in this paper by Mr. Kelcey. With the rapid development of motor traffic on the highway, increased speeds, necessary changes in roadway design, and other factors, this problem is as important to safe operation of highway vehicles as double tracking was in the development of railways.

The author emphasizes the interesting and important relationship, in so far as motor-vehicle traffic is to be controlled effectively, between psychology and engineering. This relationship has become vitally important in regulating the fundamental factor in highway transport—namely, the driver. In the serious situation with which society is now confronted in the annual loss of thousands of

[8] Strathcona Prof. of Eng. Mechanics, Yale Univ.; Pres., Eno Foundation, Saugatuck, Conn.

[8a] Received by the Secretary January 31, 1940.

lives and millions of dollars, this cooperation between psychologists and engineers is increasingly desirable.

In the paragraph with the caption "Heads Up," there is brought to mind a query current some years ago in casual groups discussing highway safety. It is, perhaps, of more value in provoking thought than in any definite answer likely to be given. The question was: "How far ahead of the car do you drive when operating on the highway?" The answer, of course, depends on the individual driver, the visibility of the road, the speed, and other factors. The writer tried this question on a number of drivers and several times received the answer: "Why, not at all. I drive where I'm sitting"—or words to that effect; but on further consideration there was always the admission that the driver does look ahead and acts, or is prepared to act, as indicated by what he sees. The actual distance when a concrete estimate was asked for would vary from a few yards in front of the car to "as far as I can see."

The greatest advantage of channelization, perhaps, is the elimination of the head-on collision. Mr. Kelcey's study of relative speeds (Figs. 1, 2, and 3) is interesting and might be extended. Using the condition which he mentions—namely, two cars, one at 45 miles per hr and the other at 40 miles per hr—if their paths are at right angles the relative velocity of A to B would be about 60 miles per hr in the direction shown in Fig. 51. This is impressive in the case

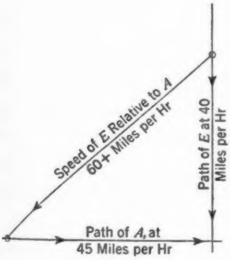

Fig. 51

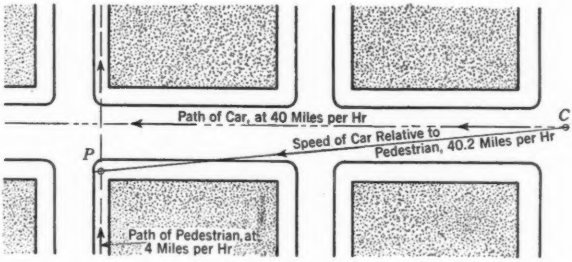

Fig. 52

of a collision of cars at a right-angled intersection, a type of accident which is almost as damaging to life and property as the head-on collision. If the speed of each car happens to be 60 miles per hr, the relative velocity, one to the other, is 85 miles per hr, a rate of travel which (as has unfortunately been demonstrated too frequently) is capable of inflicting great damage and loss of life.

Consideration of the pedestrian problem from a similar angle, even if based on hypothetical (but entirely reasonable) physical conditions, may emphasize some of these difficulties. Assume a broad street or avenue, crossed at regular intervals by others, and a pedestrian, capable of walking 4 miles per hr, whose path leads him across the main avenue (Fig. 52). Assume further an automobile approaching from the pedestrian's right at 40 miles per hr—a speed that is 10 times as fast as the pedestrian's. If the pedestrian, still on the sidewalk at the point, P, is 50 ft from the point where his path intersects that of the car, he must give careful (and prayerful) consideration to the car as it passes the point C, 500 ft away, or a matter of some two blocks. He must not only sense the

car from this distance but he must be able to appraise its speed of approach and govern his own action in crossing the street accordingly. In 1939 the Connecticut Department of Motor Vehicles made a study of highway accidents in which pedestrians had been killed and discovered the startling fact that more than 90% of such victims were not drivers—that is, had never been licensed as operators, and so, presumably, knew little or nothing at first-hand of the behavior and idiosyncracies of automobiles.

In discussing the pedestrian problem, Mr. Kelcey introduces again that combination of psychology and engineering which would seem to be a most promising form of professional cooperation. Eliminating or reducing pedestrian confusion as one definite objective in planning the physical structure of the road is entirely praiseworthy. If this is kept constantly in mind when directing channels are provided, and the channel devices are made to serve also as pedestrian refuges or isles of safety, a long step will be taken toward the goal of pedestrian safety. The pedestrian certainly has rights on the highway and, although he should exercise due care, every effort should be made to provide physical aids for his safety.

Mr. Kelcey makes reference to William Phelps Eno and his work in the regulation and control of traffic, which has extended continuously over a period of more than forty years. Indeed, the rotary system of traffic regulation which was first suggested by Mr. Eno about 1905, and described in detail as applied to Columbus Circle in New York City, is a direct ancestor of the principles which Mr. Kelcey has developed. In fact, the germ of the rotary traffic idea was embodied in "Rules for Driving," written by Mr. Eno and adopted as legal traffic regulations by the Police Department of New York on October 30, 1903. Undoubtedly, this set of traffic regulations is the forerunner of all such codes since written because, although it was compiled primarily for horse-drawn traffic before the motor vehicle became a problem in street traffic, subsequent codes, no matter how extended, have used it as a basis.

T. M. MATSON,[9] ESQ. (by letter).[9a]—Many elements are developed in this paper which should promote considerable discussion on the part of traffic and highway engineers. Mr. Kelcey is to be congratulated for his work. His efforts should not only influence the design of new roadways, but should also suggest numerous applications at existing "weak spots" in many of the roadways that have been built in the past.

It is apparent that many of the intersections in operation today are failing to carry their traffic loads safely and smoothly because of inadequate channelization mechanisms. It is clear, too, that grade intersections generally are recognized as hazardous locations. Any device that will promote facility of movement with increased safety at the thousands of non-channelized intersections in the roadway system of this nation should be a major factor in the material reduction of the nation's motor-vehicle accident record.

It is evident that traffic islands will need to be introduced into the roadway area if channelization is to be accomplished. Therefore, attention is turned to

[9] Research Asst., Bureau for Street Traffic Research, Yale Univ., New Haven, Conn.

[9a] Received by the Secretary February 14, 1940.

some of the necessary considerations that are applicable to the placing of such islands in the roadway in such manner as not to increase hazards of operation and yet promote facility of movement.

It must be recognized at the outset that there are many highway engineers to whom any obstruction of any character in the roadway is an anathema. Such feelings are not without some merit and, although the hazard evils of traffic islands are perhaps exaggerated, their virtues as channelizing mechanisms are not at all well understood or appreciated. The problem remains, therefore, as to what can be done toward gaining the advantages of channelization through the use of traffic islands and at the same time avoid the hazards which many people ordinarily associate with them.

Streamlining.—As motor vehicle speeds increase, the need for more and more "streamlining" becomes evident. Although the traffic-approach end of a divisional island may be quite blunt for slow speeds (less than 20 to 25 miles per hr) the need for literally "pulling out" the end, or tapering the approach, of such island becomes acute at high speeds. Just how long a transitional line is required per foot of lateral translation is not well understood. With speeds of 50 and 60 miles per hr, it is quite probable that the lateral translation of vehicles at a rate of greater than 1 to 60 is not too remote from what might be termed an "average natural path of translation." This is quite evident when one notices at what point a motorist begins to veer from his course in clearing an obstacle in his path. At higher speeds, even greater distances of translation are used by the motorist.

Research on this point is greatly needed. Until empirical values are determined, engineering judgment must be applied. It is probable that such values as judgment may indicate will fall far short of actual preferential paths of motorists. The need for extreme streamlining of islands under conditions of high speed cannot be stressed too strongly.

Pre-Warning.—As a complement to the streamlining of islands, attention is turned to the need for ample pre-warning of the motorist that he is approaching an island or "void" in the pavement area. This should be accomplished by means of lane markings and other appurtenances. Such center guide lines may be better designed if they begin as ordinary paint lines and increase their severity and tone of "authority" as they approach the end of the island. Thus, beginning with a "no-passing" center line, the next part of such guiding devices may be painted with the usual "cat-eye" forms of pavement buttons. This marker, in turn, would give way to a slightly raised divider strip of material which contrasts in color and texture to that of the pavement. Such a divisional strip would become rougher and more severely ridged as it approached the end of the island proper. In addition to such guiding devices, painted or embossed roadway arrows or other markings may be used in combination forms of guidance which are designed to urge the motorist into a path or channel where the island proper definitely confines the lateral movement.

Location of Approach End of Island.—In general, when a divisional island is introduced into the roadway where speeds are high, every effort should be made to place the actual "approach end" of the island slightly to the left of the apparent center of the roadway. By so doing, the approach end of the island is

outside of the normal course of the vehicular stream and additional safety results.

Splitting of One-Way Stream.—When channelization requires that a one-way stream of traffic flow be divided into two one-way streams, it is probable that different methods of island placement will be followed. What has been stated applied primarily to the splitting of two-way traffic into two one-way streams.

When a one-way stream is to be split into two one-way streams, such island of division as may be adopted should be so designed as to leave an area of pavement at the approach end of the island to provide the motorist with a "last chance" turn or decision into either of the two traffic channels. Thus there will be an area of pavement at the crotch of the two ways which will permit the motorist a final opportunity to reach the desired channel. Such pavement should be of different color and texture than that which is used in the normal channels of flow.

Adequate Visibility.—The need for adequate visibility by the motorist of the approach ends of traffic islands cannot be over-emphasized. Here there are two main factors of concern—"sight distance" and "light condition." It is obvious that, under conditions of night, either reflecting or illumination equipment is required to maintain high standards of safety.

The treatment of low-sight distances on the approach to medial divisional islands is a far more difficult problem. Where alinement or crests prohibit adequate safe sight distance, it is necessary either to improve the sight distance or to extend the medial island to a point of tangency or crest where adequate sight distances may be obtained.

Island Curbs.—Because of the importance of adequate visibility, attention is called to the present (1940) developments in curbs for islands. The State Highway Departments of New Jersey and California are experimenting with recessed faces on the island curbs. Such recesses and their materials of construction are especially designed for their light-reflecting properties.

The size and slope of the curbs to be installed for island purposes also merits much attention. There seems to be a trend toward sloping faced curbs of relatively low height. Slopes less than 45° to the horizontal are being used so that the motorist who fails to hold to his channel will not be thrown as severely; nor would his vehicle suffer the damage resulting from high, vertical curbs.

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DISCUSSIONS

PRESSURE-MOMENTUM THEORY APPLIED TO THE BROAD-CRESTED WEIR

Discussion

BY MESSRS. J. C. STEVENS, AND H. G. WILM

J. C. STEVENS,[7] M. AM. SOC. C. E. (by letter).[7a]—The results of some interesting and practical experiments are reported in this paper. The authors have presented a rational expression for the coefficient C in the well-known weir formula, $Q = C\,L\,H^{1.5}$. Their mathematical approach to this expression, however, merits further consideration. Equation (15) is orthodox and could have been written directly from an inspection of Fig. 3, provided the hypothesis is accepted that the pressure on the upstream face of the weir is that due to the depth d_1. Although this assumption appears to be confirmed by the experiments, it can be considered only an empirical approximation.

There is a downward acceleration of the water at the upstream edge of the weir and, as a falling body loses weight in proportion to its downward acceleration, to that extent is the pressure on the front face reduced below the static pressure due to the depth d_1. In other words the flow at this section is not parallel but curvilinear. This fact is well illustrated by the profile of pressure heads in Fig. 10, which is substantially below the water-surface profile in the section of the upstream edge of the weir. There is also a dynamic pressure on the upstream face of the weir due to the velocity of approach. If this velocity is uniform and equal to V_1, the dynamic pressure is $\frac{w\,b\,d_4}{g}(1 - \cos\alpha)$ when α is the angle through which the initial direction of the velocity has been turned by the obstruction. Although the front face of the weir is at right angles to the initial direction of V_1, the value of α is not 90° but something less. There is a horizontal, revolving eddy or cushion in front of the weir which deflects the water upward and over the crest, giving it an upward acceleration as it passes over the edge. This tends to balance the downward acceleration of the surface water. The combined effect is seen in the curved line of pressures against the front face of the weir in run No. 11 of Fig. 10.

NOTE.—This paper by H. A. Doeringsfeld, Esq., and C. L. Barker, Assoc. M. Am. Soc. C. E., was published in December, 1939, *Proceedings*. This discussion is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion of the paper.

[7] Cons. Hydr. Engr. (Stevens & Koon), Portland, Ore.

[7a] Received by the Secretary January 16, 1940.

Equations (13a) and (13b) are expressions of the pressure-plus-momentum functions—not forces. This function was used effectively by Julian Hinds,[8] M. Am. Soc. C. E., in his analysis of the hydraulic jump. It is also designated as the M-function by Boris A. Bakhmeteff,[9] M. Am. Soc. C. E. The authors have considered the momentum of the water as $\frac{w\,Q\,V}{g}$, whereas if the mean velocity is used a coefficient should be applied. The value of this coefficient depends upon the velocity distribution within the prism of flowing water. It is given by[10]

$$C_m = \frac{\int_0^A v^2\,dA}{V^2\,A} \quad \text{(20)}$$

Equation (9) and its preceding development appear to have but slight relation to the remainder of the paper, which is fortunate, for it contains several errors, some of which are doubtless typographical.

The pressure intensity at any point on a sloping bed supporting a stream is $p = w\,d'\cos\theta$, in which d' is the depth normal to the bed, or $p = w\,d\cos^2\theta$ in which d is the vertical depth. The total pressure force in a unit width for the two cases then becomes

$$P' = w\,\frac{(d')^2}{2}\cos\theta \quad \text{(21a)}$$

and

$$P = w\,\frac{d^2}{2}\cos^2\theta \quad \text{(21b)}$$

Writing the momentum equation in terms of depths normal to the channel bed and velocities parallel thereto, as given in Fig. 2, the expression for a unit width is

$$\left[\frac{(d_1')^2}{2}\cos\theta + C_{m1}\frac{q\,v_1'}{g}\right] + \frac{d_1' + d_2'}{2}\,\Delta l'\sin\theta - \left[\frac{(d_2')^2}{2}\cos\theta + C_{m2}\frac{q\,v_2'}{g}\right] = 0 \quad \text{(22)}$$

in which C_m is the coefficient of momentum, and q is the flow per unit width.

This may be expressed in terms of vertical depths and horizontal velocities by substituting: $d' = d\cos\theta$; $\Delta l' = \frac{\Delta l}{\cos\theta}$; $v' = \frac{v}{\cos\theta}$; and $q = v'\,d' = v\,d$. After dividing by $\cos^2\theta$:

$$\left(\frac{d_1^2}{2} + C_{m1}\frac{v_1^2\,d_1}{g\cos^3\theta}\right) + \frac{d_1 + d_2}{2}\,\Delta l\,\frac{\tan\theta}{\cos\theta} - \left(\frac{d_2^2}{2} + C_{m2}\frac{v_2^2\,d_2}{g\cos^3\theta}\right) = 0 \quad \text{(23)}$$

which is seen to differ materially from Equation (9). However, for $\theta = 0$, it reduces to the first two terms of Equation (15), since $v^2\,d = \frac{Q^2}{b^2\,d}$.

[8] "The Hydraulic Jump and Critical Depth in Design," by Julian Hinds, *Engineering News-Record*, Vol. 85, 1920, p. 338.

[9] "Hydraulics of Open Channels," by Boris A. Bakhmeteff, McGraw-Hill Book Co., Inc., New York, N. Y., 1st Ed., p. 234.

[10] "Applied Fluid Mechanics," by M. P. O'Brien, M. Am. Soc. C. E., and George H. Hickox, Assoc. M. Am. Soc. C. E., McGraw-Hill Book Co., Inc., New York, N. Y., 1st Ed., p. 271.

This expression neglects the friction which is an upstream force. If d_2 is less than d_1, there is a downward acceleration; hence the unit weight is not w, but $w\left(1 - \frac{\alpha}{g}\right)$, in which α is the downward acceleration. However, as long as the upstream friction force is neglected, the specific weight cancels out as does the width and Equation (23) is correct for any sloping rectangular channel.

The error of omitting $\cos\theta$ in $p = w\, d \cos\theta$ amounts to 1% for an 8° slope and 14% for 30°. Therefore, it must not be omitted in steep chutes and on the face of overflow dams. Wherever the slope is steep enough to take into account the force of gravity as expressed by the last term of Equation (9), then the effect of this slope on the static pressure in the vertical should also be considered as included in Equation (23).

As a metering device, the broad-crested weir may have advantages under some circumstances. The rounding of the upstream curve will probably have the effect of increasing the discharge. The deposit of sand, silt, or sludge against the upstream face will also increase the discharge, since it lessens the upstream reaction to the dynamic force against the front face. It is not applicable without further tests where the tailwater is likely to submerge the crest more than $\frac{2}{3} H$.

The Parshall flume[11] has decided advantages over this type of weir because it keeps itself generally free of sediment and the calibrations cover sizes from 3-in. to 50-ft crest widths and include both free-flow and submerged-flow conditions. For the former, as in this case, only a single head is observed up to a two-thirds submergence. Beyond that limit, of course, both headwater and tailwater levels must be observed, for which the flows are given in empirical formulas and tables.

There is need for an open-channel metering device which is indexed by far smaller heads than is possible with any type of free-flow weir. The writer believes that the submerged Parshall flume and the submerged orifice[12] are the best devices so far developed to meet such conditions, largely because of the exhaustive calibrations they have received.

H. G. WILM,[13] ESQ. (by letter).[13a]—Many attempts have been made to develop devices for water measurement which could be "rated" on a purely theoretical basis, without the need for empirical coefficients to adjust theory to actuality. Until now, as far as the writer knows, no such attempt has been entirely successful because of errors in the basic assumptions involved. One assumption which, perhaps, results in the most deviations of theory from practice, is that water is without viscosity and hence without friction. Another is that acceleration, deceleration, and changes in direction of water flow are accomplished without expenditure of either time or space.

Fig. 3 and the authors' design for the broad-crested weir (Fig. 10) used in

[11] *Bulletin No. 423*, Colorado Experiment Station (U. S. Dept. of Agriculture), Ft. Collins, Colo.; see also "The Improved Venturi Flume," by Ralph L. Parshall, Assoc. M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. 89 (1926), p. 841.

[12] "The Calco Metergate," The R. Hardesty Manufacturing Co., Denver, Colo.

[13] Silviculturist, Rocky Mountain Forest and Range Experiment Station, Fort Collins, Colo.

[13a] Received by the Secretary January 25, 1940.

their experiment illustrate partial failure to recognize empirical requirements. Throughout the paper, apparently, the assumption is made that parallel flow will occur over the level weir surface, except where the flow is influenced by end conditions. This condition is impossible to obtain in practice. For any given discharge the depth at any point along the weir will depend on the friction of the weir material and on the distance of the point in question from the ends of the weir.

The authors' use of a weir with a sharp upstream corner was due, apparently, to the second of the foregoing assumptions. Since water does require time and space in which to accomplish a sudden change in direction of flow, the inevitable result is flow separation just above and downstream from the sharp corner. The threads in Fig. 8 illustrate this well-known phenomenon. Its significance to their study is simply that discharge over the weir is necessarily influenced by such separation. The actual depth influencing discharge over the weir is not exactly "d_3" in Fig. 3, but an indefinite depth, measured above the bottom of the *vena contracta* rather than above the weir surface.

As a combined result of these two factors, the "minimum depth" (d_3) for any given discharge is necessarily an uncertain quantity, depending for its exact magnitude and location upon velocity of approach, rate of discharge, surface friction, and weir length. Although the resulting errors in measurement of discharge might not be excessive, they should easily be as great (up to 8% or more, depending on the value of "K") as those observed by Messrs. Doeringsfeld and Barker. Since other flow-measuring devices of greater accuracy are available, and since field installations ordinarily show greater variability than laboratory studies, errors of such size would invalidate the authors' method for most purposes.

Professor Woodburn,[14] in the publication cited by the authors, discusses in detail the locations of critical depths on the weirs studied, as follows:

> "The foregoing discussion of the position of critical depth pertains to the weirs in which the entrance was rounded. With a square-cornered entrance the conformation of the water surface over the weir was quite different, as is seen by comparing the profiles of Series *A* with those of Series *B* to *G*. The initial drop in the water surface was greater with the square-cornered entrance than with the rounded entrance preceding a level crest 10 ft. broad. With the square-cornered entrance the water surface dropped to a point below the critical depth and then commenced to rise gradually. This rise continued either until it approached the critical depth, when the surface became wavy and turbulent, or until it reached the vertical curve caused by the fall. The critical depth was reached and waves were formed only with flows of less than about 4 cu. ft. per sec. The waves formed with the rounded entrance were smooth in profile, and the water surface was practically level in cross-section at all points. With the square-cornered entrance the waves reached more of a peak, being considerably higher in the center of the stream than at the sides; and there was more surface turbulence."

Professor Bakhmeteff,[15] in his discussion of Professor Woodburn's paper,

[14] "Tests of Broad-Crested Weirs," by James G. Woodburn, *Transactions*, Am. Soc. C. E., Vol. 96 (1932), pp. 401–404.

[15] Discussion by Boris A. Bakhmeteff of "Tests of Broad-Crested Weirs," by James G. Woodburn, *Transactions*, Am. Soc. C. E., Vol. 96 (1932), pp. 423–434.

effectively clarifies the relation of critical depths and associated phenomena to flow over broad-crested weirs.

Fig. 10 in the paper illustrates, in two ways, the influence of the weir employed upon flow characteristics. First, plotted pressures show violent variation directly above the sharp entrance corner. Wherever curved flow exists, of course, pressures will deviate from depths. The variation shown in Fig. 10, however, is doubtless caused by sharply curved flow at the bottom as well as at the top of the flow column, with the column of actual flow moving an appreciable distance above the weir surface and with a body of relatively dead water between. Second, the "minimum depth" (d_3) is well below both computed and actual critical depths in both illustrated cases. Since the weir surface is level longitudinally, this condition is a purely local one induced by residual momentum and pronounced flow curvature in the entrance zone. If the weir had been longer, flow depths (as described by Professor Woodburn) would have increased downstream from the zone of curvature until they reached characteristic rippled flow at the critical depths. As concluded by the writer and his collaborators,[16] however, flow on a level broad-crested weir is sufficiently unstable to be unsatisfactory for purposes of discharge measurements.

These arguments are by no means intended to indicate that the broad-crested weir is unsuited to measurement of open-channel discharge. This type of structure is excellently adapted to the purpose and should measure discharge with accuracy equal to any other type of device suited to field use. Furthermore, when properly designed and constructed, broad-crested weirs can be calibrated accurately on a rational basis. In order to do so, however, several requirements must be met in design:

(1) All changes in flow direction upstream of the measuring point must be accomplished by well-rounded entrance transitions. The necessary minimum radius of curvature varies with expected velocities of approach.

(2) The point of pressure or depth measurement must lie in a zone of relatively straight-line flow, where pressures and depths show close agreement. In ordinary design, this means that the weir length must be at least three times the greatest water depth to be measured—preferably four to six times—and the point of measurement should be near the longitudinal midpoint of the weir.

(3) The weir surface should have a longitudinal slope sufficiently greater than the critical slope for the weir material so that at all stages rapid, stable flow velocities will occur at the point of measurement. Critical slopes for any weir material may be computed by the following formula:

$$S_c = \frac{g}{C^2} \times \frac{P}{b} \qquad (24)$$

in which S_c is the critical slope; g is acceleration due to gravity; P is the wetted perimeter; b is the top width; and C is the Chézy coefficient corresponding to a given Kutter's friction factor and depth of flow.

(4) The Kutter's friction factor must be known within close limits for the weir material employed. This can be determined either by a simple hydraulic-

[16] "Measurement of Débris-Laden Stream Flow with Critical-Depth Flumes," by H. G. Wilm, John S. Cotton, and H. C. Storey, *Transactions*, Am. Soc. C. E., Vol. 103 (1938), pp. 1247–1249.

laboratory test or by obtaining a few measured rating points for a field installation.

With satisfaction of these requirements the broad-crested weir affords a simple and easily calibrated device for measuring flow, and one which is relatively free from influence by velocity and stage conditions either upstream or downstream of the structure. If the weir surface is laid on a longitudinal slope approximately equal to the critical slope for the range of depths employed, discharge should conform with reasonable closeness to the theoretical formula for flow at critical depths in a rectangular open channel:

$$Q = \sqrt{g}\, b\, h_c^{1.5} \qquad (25)$$

The weir must be long enough, however, to permit establishment of stable straight-line flow, with the point of measurement within the zone of stable flow and downstream from the zone of transition influence. Since such flow may exhibit some instability, expressed in surface waves, it is safer to give the weir a slope greater than critical. In this case, a calibration formula may be derived by the method described by Fred C. Scobey,[17] M. Am. Soc. C. E., and outlined by R. L. Stoker.[18] One modification only is suggested, based on experimental observations—that the location of critical depths be assumed to lie along a line, not vertical, but inclined downstream at an angle of approximately 10° from the vertical. The origin of the line should be fixed at the upstream edge of the broad-crested weir (not including any rounded entrance).

When surface curves for several discharges have been computed by this method, depths may be taken at any desired point of measurement and a calibration formula computed by fitting a logarithmic straight line to the stage-discharge relation. This formula will comply with the form

$$Q = C\, b\, h^{1.5} \qquad (26)$$

in which C is an empirical coefficient to correct for friction losses and for progressive downstream displacement of the critical depth with increasing discharges.

The writer, with others, worked from 1934 to 1938 on the development of a flume for use in measuring open-channel flows containing bed loads. This flume functions on the same principle as a broad-crested weir. The essential difference in design involves only the use of horizontal transitions instead of a bottom contraction to accomplish the development of critical flow. Pressure measurements are taken within the flume, at a point downstream from the zone of critical depths. Since the floor slope is greater than the critical, stable scouring velocities are maintained at the measuring section.

Preliminary results of these studies were published in 1938.[19] Further developments by means of half-scale model studies, as they pertain to the present discussion, may be summarized as follows:

[17] "The Flow of Water in Flumes," by Fred C. Scobey, *Technical Bulletin No. 393*, U. S. Dept. of Agriculture, 1933.

[18] Discussion by R. L. Stoker of "Measurement of Débris-Laden Stream Flow with Critical-Depth Flumes," by H. G. Wilm, John S. Cotton, and H. C. Storey, *Transactions*, Am. Soc. C. E., Vol. 103 (1938), pp. 1268–1270.

[19] "Measurement of Débris-Laden Stream Flow with Critical-Depth Flumes," by H. G. Wilm, John S. Cotton, and H. C. Storey, *Transactions*, Am. Soc. C. E., Vol. 103 (1938), p. 1237.

(1) Variations in wall and floor friction from a Kutter's factor (n) of 0.010 to one of 0.015 cause an inverse variation in discharge up to 5%.

(2) Discharges appear to be little affected by variations in approach and exit conditions. Tailwater is tolerated without appreciable effect on discharge, up to 70% to 80% of the initial specific energy of the discharge. (Initial specific energy equals depth plus velocity head, considering the channel bed at flume entrance as zero head.)

(3) For flume lengths shorter than a minimum of twice the distance from entrance transition to piezometer (about six times the greatest measured water depth), the depth-discharge relation changes rapidly. Increase in length of the downstream section, up to a maximum overall length of three times the minimum, appears to have no measurable effect on discharge.

(4) Comparisons were made of the relative accuracy of several means of transmitting pressures to piezometer wells: Static tubes, standard piezometer orifices, and vertically elongated slots in the flume wall. Compared to static-tube data as a check, the slots tested seem to be as reliable as pressure orifices. Variation in width of slot (in the models) from $\frac{1}{32}$ to $\frac{1}{8}$ in., and in wall thickness from $\frac{1}{8}$ to $\frac{1}{2}$ in., appeared to cause no variation in measured pressures. Within the roughness range tested (n = 0.010 to 0.015), flume-wall roughness has little or no effect upon orifice or slot readings.

All of the aforementioned studies were conducted during the period 1934–1938, while the writer was a staff member of the California Forest and Range Experiment Station, under its auspices and in cooperation with the University of California, at Berkeley, Calif. They were designed to test and calibrate a particular flume design—that for the San Dimas Flume in its final form as illustrated by Mr. Stoker.[20] The principles involved and the results of these studies, however, should be applicable to any broad-crested weir that conforms in design and construction to the foregoing requirements.

As an illustration, Table 4 presents observed depth and discharge data from model studies of the San Dimas Flume, with computed depths obtained by the aforementioned specific-energy method.

The flume involved was the 6-in. model described by Mr. Stoker. Observed depths were obtained by point gage to the nearest 0.001 ft; discharges were measured by means of 4 by 2 in. and 6 by 4 in. venturi meters. For observations Nos. 106, 112, 114, 115, 125, 129, 130, and 131 the flume was of smooth, shellacked and varnished wood, with a friction factor (n) estimated at 0.010. For the other observations given in Table 4, the same flume was roughened with two different sizes of sand. In these cases the value of n was computed by the formula

$$n = 0.043\, D^{1/6} \qquad (27)$$

in which D = mean diameter of sand particle, in feet. The floor slope of the flume was 0.031 for the first four observations, and 0.050 for the others.

It will be noted that most of the deviations of computed from observed depths are positive in direction. This should be due either to a slight error

[20] Discussion by R. L. Stoker of "Measurement of Débris-Laden Stream Flow with Critical-Depth Flumes," by H. G. Wilm, John S. Cotton, and H. C. Storey, *Transactions*, Am. Soc. C. E., Vol. 103 (1938), p. 1269.

in the formula for computing n, or to the incorrect location of critical depths in plotting water-surface curves. However, since flow acceleration is relatively slow within these flumes, considerable longitudinal displacement of the line of

TABLE 4.—Comparison of Observed Depths with Depths Computed by Specific-Energy Method; Six-Inch Model of San Dimas Flume

(Computations Were Made by Slide-Rule)

Observation No.	Floor slope, S, in feet per foot	Kutter's friction factor, n	Observed discharge, in cubic feet per sec	Depths, in Feet			Percentage deviation
				Observed	Computed	Deviation	
106	0.031	0.010	0.179	0.116	0.115	−0.001	−0.9
112	0.031	0.010	0.418	0.223	0.223	0.000	0.0
114	0.031	0.010	0.630	0.301	0.303	+0.002	+0.7
115	0.031	0.010	0.923	0.397	0.399	+0.002	+0.5
125	0.050	0.010	0.180	0.107	0.107	0.000	0.0
129	0.050	0.010	0.420	0.205	0.207	+0.002	+1.0
130	0.050	0.010	0.633	0.281	0.284	+0.003	+1.1
131	0.050	0.010	0.917	0.374	0.378	+0.004	+1.1
166	0.050	0.0134	0.180	0.112	0.112	0.000	0.0
168	0.050	0.0134	0.424	0.215	0.220	+0.005	+2.3
169	0.050	0.0134	0.640	0.293	0.298	+0.005	+1.7
191	0.050	0.0152	0.181	0.120	0.121	+0.001	+0.8
189	0.050	0.0152	0.420	0.219	0.222	+0.003	+1.4
187	0.050	0.0152	0.637	0.298	0.299	+0.001	+0.3
188	0.050	0.0152	0.926	0.393	0.402	+0.009	+2.3
				Average percentage deviation			= ±0.94.

critical depths may occur without causing appreciable error in measured depths at the piezometer.

Corrections for *Transactions:* In Fig. 2, d_1 and d_2 are the vertical depths of flow at points 1 and 2 rather than the vertical components of d_1' and d_2', respectively; in the line following Equation (7), change "P" to "p" and correct the definition of Δl to read "$\Delta l = \Delta l' \cos \theta$"; in Equation (8) change "P" to "p"; in the line preceding Equation 13(a) change "force" to "function"; change Equation (15) to read

$$“\left(\frac{d_1^2}{2} + \frac{Q^2}{g\, b^2\, d_1}\right) - \left(\frac{d_3^2}{2} + \frac{Q^2}{g\, d_3\, b^2}\right) - \left(\frac{2\, d_1 - d_4}{2}\right) d_4 = 0”;$$

and invert Fig. 5.

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DISCUSSIONS

STANDARDS OF PROFESSIONAL RELATIONS AND CONDUCT

Discussion

BY MESSRS. LOUIS E. AYRES, IVAN C. CRAWFORD, WALTER H. WHEELER, CHARLES R. GOW, J. T. L. MCNEW, AND W. L. WATERS

LOUIS E. AYRES,[6] M. AM. SOC. C. E. (by letter).[6a]—This paper presents a compilation of the best thoughts from many existing codes, leavened and supplemented by judgment and experience. The author has gone beyond the scope of the title, and, as stated, he "discusses the vital relation of good principles and good conduct to success in life." Much of this material should be helpful to young men in all professions, as it is essential to right living and sound happiness. The author is fundamental and "old-fashioned" in his approach to the problems of life. He still believes in the virtues of courage, character, hard work, personal energy, and will power; and he discounts the validity of some modern theories of psychologists which tend to excuse failure and to discount success on the basis of heredity or environment. One must conclude that the author's thesis is one that will not be accepted widely in an easy-going generation when so many seek success without effort, and as their due; but it is at such a time that the reiteration of fundamentals becomes important.

In reading the paper one is impressed by its scope and detail. In fact, it is easy to be over-impressed by the enumeration and comments on so many essentials to success. How can the ordinary individual hope to measure up to the standards set forth by the author? Is there not a possibility that many a younger man, after conscientious study of this document, may be discouraged by the requirements of a professional career? Obviously, the point to emphasize is that all codes, like the Ten Commandments and the Sermon on the Mount, are the ideal objectives to be sought but not the ends that are easily or immediately achieved. Paradoxical as it may sound, success itself consists in

NOTE.—This paper by Daniel W. Mead, Past-President and Hon. M. Am. Soc. C. E., was published in January, 1940, *Proceedings*. This discussion is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion of the paper.

[6] Cons. Engr. (Ayres, Lewis, Norris & May), Ann Arbor, Mich.

[6a] Received by the Secretary January 19, 1940.

acquiring those very elements in character and personality that lead to success. In other words, the road is open to him who has the will power to persevere in his own self-improvement. The author's paper then becomes a textbook for the student, and a practical guide for any one who would keep in mind the "rules of the game" if, in the end, life is to reward him to his satisfaction.

The author calls attention to the importance of budgeting one's time to avoid the often wasted hours, and suggests that one half of the out-of-office time of the young engineer may well be devoted "to study that is collateral to his job." What kind of study is "collateral" to an engineer's job? Is such study to be confined solely to technical fields? Is it not equally or even more important that the young engineer make an early and systematic start in the reading of good literature, history, and other fields that will broaden his interests. It is also desirable that engineers become active as citizens and that they participate in politics and other group activities in their communities. They are also urged to be active in engineering organizations—local, state, and national. With such a wide range of demands on the out-of-office time of the engineer, the budgeting of his excess energy becomes of prime importance. It is absolutely essential, therefore, that he choose with discretion among the many activities that intrigue his interest, to the end that effort may not be spread too thin, or health injured by over-strenuous living.

The writer would like to suggest the inclusion of one additional section having to do with the "Ethics of the Expert Witness." The engineer is frequently called upon to give testimony in courts or before commissions and in so doing often finds himself under pressure to be a partisan, to take the "side" of his client, and to present figures and express opinions that are biased and either more or less than the truth. He is the victim of a system which depends for results on the presentation of extreme viewpoints, often to the extent of falsity, on the theory that one half of the sum of zero and infinity results in a real integer. The Code of Ethics of the Society states (29)[6b] that it shall be considered unprofessional for an engineer "To advertise in self-laudatory language." In the light of this rule, who can but deprecate the extremes to which expert witnesses sometimes permit themselves to be led by an attorney in court in presenting their experience record and their qualifications for the expression of expert opinion? How frequently the balloon of their extravagant claims is punctured by the opposing attorney on cross-examination. Obviously, these statements are not to be interpreted as any general accusation of engineers as expert witnesses. As a rule, it may be asserted, they do conduct themselves in court on a relatively high plane of professional conduct; but the exceptions are sufficiently numerous to cause an opinion to prevail that almost any claim can be supported in court by an expert.

The writer is not prepared to suggest the complete statement of principles that should be incorporated with this paper to cover the proper conduct of expert witnesses. One may hope that Professor Mead will be willing to do so. It has long been the opinion of the writer that there should be an addition to the official Code of Ethics (29) approximately in the following language:

[6b] For reference to numerals in parentheses, see "Bibliography of the Literature on Ethics and Human Engineering," in the Appendix of the paper.

It shall be considered unprofessional and inconsistent with honorable and dignified bearing for any member of the American Society of Civil Engineers:

8. To give expert testimony in court, or before a commission, differing in substance or implication from what he would present and defend before a technical meeting of this Society.

The writer would close this brief discussion by a word of emphasis on service as a motivating factor in human action if the satisfaction of success is to be fully realized.

The word "service" has a hackneyed flavor. It has been overworked. It suggests to many minds mainly social service work in the slums and among the poor and the unfortunate. In its true sense, however, it comprehends all actions that preserve and develop civilization. It is the essence of the Golden Rule (26). It should motivate man's actions toward the family, the church, and the state. The first objective of the employer to the employee, the employee to the employer, and the engineer to his client should be service. The engineer's interest in technical societies should be based on his desire and willingness to serve, and it goes without saying that this great Society has but one prime objective—namely, to be unselfishly useful to the profession and to the public. The criterion of service should be ever present in all the acts of the Society members, both those who aim at technical improvement and those concerned with the personal welfare of members. What other justification can there be, for example, for registration of engineers other than the ultimate benefit to the public whom the engineer aims to serve?

Considerable thought has been given in the past to the preparation of arguments that may convince young men that they should join the Society. One may point out the value of the publications, of meetings, and of personal contacts with successful engineers. Perhaps such matters as jobs, salaries, and wages may also be presented as inducements; but, if the thought of service is omitted from the arguments, a most important item has been overlooked. Young men, in particular, may be appealed to on this basis; and older men, who have not become hardened and cynical, generally accept this theory of living as the one most conducive to success and real happiness. A proper emphasis on the ideal of service is desirable, therefore, in this statement of "Standards of Professional Relations and Conduct."

Ivan C. Crawford,[7] M. Am. Soc. C. E. (by letter).[7a]—In completing and placing before the engineering profession this valuable compilation and interpretation of the ethics which should govern engineers in their relationships, one with another, and with their clients and the public, Professor Mead has accomplished a most worth-while task. Furthermore, he brings the subject vividly and concretely to the engineering student and the young engineer. Particularly pleasing is the emphasis placed upon personal energy and will power as essential elements for success, and the necessity for the student and the young engineer to budget leisure time carefully.

[7] Dean, School of Eng. and Architecture, Univ. of Kansas, Lawrence, Kans.

[7a] Received by the Secretary January 18, 1940.

As the author has stated, in general, the ethical principles which he presents are axiomatic. Some educators will disagree, however, with his statement (heading, "I. The Engineering Student and the Young Engineer," paragraph 5) that "On the other hand, he should not strive too much for originality." These persons will argue that engineering students particularly have been, and are, subjected to the type of training that does not encourage originality, and places a premium on the following of textbook material. Such criticisms no doubt have some foundation, but the fact still remains that it is difficult for a student or an inexperienced engineer to be original when his knowledge and experience do not give him the necessary familiarity with his field to warrant a departure from what is considered to be accepted good practice. Professor Mead's discussion of this point seems to be thoroughly sound.

Of especial interest is the section of the paper devoted to "III. Public Relations of the Engineer." Surely every one will agree with the statement (paragraph 6) that "The engineer should discourage, in every legitimate manner, the construction of public works that are economically unsound for the community, state, or nation." However, the determination of economical unsoundness will often be a matter of considerable difficulty, although, as demonstrated within the past few years, the outstanding examples of such unsoundness can usually be shown to the public. Much credit should be given to the members of the Society who have expressed their opinions fearlessly regarding unworthy projects, and who in certain cases have contributed to the abandonment of economically unsound developments.

This paper, because of its completeness and clarity, as well as the importance of the subject matter, could well be incorporated in engineering administration courses or in other courses that deal with the subject of professional ethics.

WALTER H. WHEELER,[8] M. AM. SOC. C. E. (by letter).[8a]—"Professional Conduct" is a subject of vital importance to the engineering profession. May the day come speedily when the profession will be guided by such high standards as the author has outlined so completely. At present, the profession seems to be far from that goal.

The writer agrees heartily that it is unprofessional to accept engagements for which remuneration is too low, the standard being that which was approved by the Society (32)[8b] in September, 1930. He also agrees that it is unprofessional and not in the public interest to compete for engagement on the basis of the price for services. Unfortunately, competition on that basis is far from unknown, particularly in connection with engagement of engineers for public projects.

There is another type of competition which the author did not mention and which the writer believes is both legitimate and stimulating in its effect. This type of competition is provided for to a limited extent by a bill introduced in the Congress of the United States on January 3, 1940, by the Hon. John G. Alexander of Minnesota (33). One section of the bill authorizes any

[8] Designing and Cons. Engr., Minneapolis, Minn.

[8a] Received by the Secretary January 19, 1940.

[8b] Numerals in parentheses, thus: (32), refer to corresponding numbers in the Bibliography of the Appendix of the paper, and at the end of this discussion.

bureau or department of the government to employ engineers in private practice to prepare alternate designs for the larger projects on a semi-contingent basis. Such procedure does not displace or attempt to displace the engineer of the project. It opens the way for enterprising engineers, who know how to compute costs in detail and to design so as to get the lowest construction costs without reducing strength, durability, or utility or affecting appearance, to cooperate with the engineer of the project by submitting alternate designs on a contingent basis. The payment of the engineer's fee is contingent upon showing a substantial saving by the adoption of the alternate design. The saving is determined by contractor's bids. Thus the engineer of the project may render better service to his client by reducing the cost of the construction, and the engineer who submits the successful alternate has the opportunity to realize on his ingenuity. Encouragement is given to enterprise, ingenuity, and progress. It cannot be justly claimed that there is anything undignified or unprofessional in such a procedure. There is the further consideration, as Professor Mead has stated, that engineering is a business as well as a profession.

The author's comments upon educational requirements for engineers are interesting, and particularly his statement (see heading "Introduction: Technical Training and Technical Ability"), "that the study of professional subjects in any great detail is inadvisable, because all such subjects are changing rapidly and few schools can keep in step with those changes." This is worthy of the most careful consideration by educators. The writer believes that preparation for a professional career in engineering should include more general education of a cultural nature. Thorough grounding in the fundamentals of English, public speaking, government, economics, and the law of contracts, as well as in mathematics and the sciences, is desirable. Why not enlist the aid of men engaged in private practice to lecture to students on professional subjects?

The author has referred briefly to patents. The patent laws of the United States have been one of the most stimulating causes of technical progress. The way of the inventor is not an easy one. Comparatively few inventions ever achieve commercial importance. Some of the greatest engineers have also been inventors, and the results of their efforts have been of tremendous benefit to mankind. Civilization owes much to them, and the profession should take pride in their accomplishments and should encourage them.

The profession is indebted to Professor Mead for his excellent presentation of this subject.

(32) Manuals of Engineering Practice, No. 5, p. 8.

(33) H. R. Doc. No. 7635; see *Civil Engineering*, January, 1940, p. 43.

CHARLES R. GOW,[9] M. AM. SOC. C. E. (by letter).[9a]—The several concepts, principles, and rules of conduct so well enunciated by the author of this excellent presentation might well be applied indiscriminately in the case of every young man entering upon his life's career, whether it be in the field of professional, business, or public service. In general, they constitute fundamental requisites for success in life and for this reason are entitled to most serious consideration.

[9] Pres., Warren Bros. Co., Cambridge, Mass.

[9a] Received by the Secretary January 23, 1940.

A grave peril confronts the people today because of the modern tendency to emphasize the importance of individual rights and privileges to the greater or lesser exclusion of consideration for such reciprocal essentials as duties and obligations. Economic progress and human well-being have made startling advances over the past one hundred and fifty years, largely by virtue of the willingness of the people to accept sacrifice, hardship, and the exercise of patience as the cost of such blessings. In recent times, however, there has developed a tendency to decry the success of those who heretofore have profited materially from strict adherence to old-fashioned principles, and to insist that the fruits of their efforts and sacrifices should be shared generously with their less fortunate contemporaries, including the indolent, the inefficient, and the wastrels of society.

It is undoubtedly true that a professional calling requires a somewhat more rigid code of ethical behavior than might be deemed essential in some other walk of life. The obligation of the engineer toward his client, for example, partakes, no doubt, of a more confidential character than that which exists, say, in the business world between the ordinary seller and purchaser. Men of questionable character have been known to prosper for considerable periods in the business world; but such instances have been extremely rare within the engineering profession since, as a group, its members have been too jealous of its good name to tolerate the presence even of "a single black sheep within its fold."

It is relatively easy to compile and to suggest rules of conduct for the general enhancement of the standing of a profession. Unfortunately, however, nature has endowed mankind with such a variety of diverse traits of character that complete cooperation in support of any proposed code of behavior is difficult to obtain, even among the members of relatively small and select groups. However, it is not altogether impossible in a cooperative society to subject such human frailties to a certain measure of control and restraint through the medium of effective educational measures. At one time or another most engineers have had occasion to subordinate their natural impulses in favor of another course because their intelligence has convinced them that self-discipline and cooperative action will be likely, in the long run, to secure greater rewards. In fact, in the absence of such a realization among intelligent human beings, there could be no civilized society.

Therefore, it would appear to be largely a matter of educating the individual to a full appreciation of the necessity and desirability of conforming to uniform standards of behavior which have come to be generally recognized as representing the consensus of opinion among experienced men engaged in a similar calling. This suggests the desirability of inculcating these principles in the minds of embryo members of the engineering profession at the earliest possible period in their careers. It seems somewhat incongruous to delay such efforts until the individual engineer has attained some degree of prominence in his work; yet it is the writer's observation that relatively few become seriously impressed with the importance of this subject until the magnitude of their responsibilities brings them into direct contact with it.

Should it not then become the aim of technical schools and colleges to introduce the subject of professional standards into their curricula as a component

element in the students' educational equipment? If such a code of practices and relations as that outlined by the author, or with such modifications or amplifications as might finally be determined upon by the profession in general, could be provided as a basis of a college course in practical ethics, it should be possible to produce student graduates who would be as well grounded in their understanding of conventional professional practice as they now are in technical knowledge.

Several years ago the writer was induced to accept, for a limited term, a newly created chair in "Humanics" at the Massachusetts Institute of Technology, at Cambridge, Mass., and to set up the course of instruction pertaining to the subject. The purpose of the course was to advise the students respecting the nature of the many human problems, which they might expect to encounter in their subsequent relations with other individuals, growing out of their employment experiences. Since it was a pioneer effort in an unknown field, some doubt was entertained as to the degree of interest such a course would elicit from the student body. It soon became apparent that there was an enthusiastic desire among them to acquaint themselves with all phases of the subject. The course has now been in existence more than ten years, and the writer is advised that it continues to be one of the most popular subjects offered.

This experience leads the writer to believe that any subject that is likely to affect the student's future success in his subsequent professional life will make a strong appeal to him during the educational period. There is surely no better time or opportunity to implant fixed principles of conduct in his mind and thus to fit him in advance for an automatic adjustment to the accepted ethical code of his future associates and contemporaries.

It is the writer's opinion that Professor Mead's suggested set of standards constitutes an excellent starting point for the establishment of the recognized principles which make an acceptable basis not alone for the guidance of practicing engineers but quite as well for the initial instruction of the engineering student.

J. T. L. McNEW,[10] M. AM. SOC. C. E. (by letter).[10a]—Someone has said: "The young have been preached to, lectured to, taught, exhorted, advised. They have seldom been talked to." In this paper the author has done an admirable job of "talking" ethics to young men in a straightforward manner such as to serve as an inspiration to them. The paper was needed urgently by the entire profession to fill a vacant spot in the training of young men. The tenets expounded are all based upon honesty, industry, study, perseverance, self-control, tenacity of purpose, and righteousness—the values of which, in life, are not debatable.

As a teacher of classes of senior engineers in a course in professional relations for a number of years, the writer agrees with Professor Mead as to the need that young men have for information of this kind. The only question the writer would ask is, "Why is not the paper even more inclusive?" and he realizes that great length decreases value.

[10] Prof., Highway Eng., Agri. and Mech. Coll. of Texas, College Station, Tex.

[10a] Received by the Secretary January 29, 1940.

The writer is thoroughly convinced that an overwhelmingly large majority of engineering students desire earnestly to follow their chosen professions in such a way that they may be better engineers in every sense of the word than their predecessors. They are inherently honest and are anxious to "play the game by the rules," not because of the rules, but because it is right that they so "play." They are much interested in statements of right policy as well as in the viewpoint that makes the policy or ethics right. To insure that they are privileged to obtain such information in the proper way is the obligation of teachers as well as practitioners. In an overcrowded curriculum, however, it has been, and is, an easy matter to neglect stressing the value of righteous professional conduct, although it is always admitted that failure to conform to the recognized ethical practice is the surest way to fail.

To the successful engineer, ethical practice becomes habit and habit comes from training—by doing the same thing correctly over and over again until it appears that the right way is the only way an assignment can be done. No better place exists for developing the habit of good professional conduct than on the campuses of American colleges where four years of "practice living" can be had and where mistakes may be corrected in ways other than the wrecking of an entire life. Correct habit depends upon self-discipline and upon the ability to impose self-denial at times. Literally, one may sow acts and reap habits, while the sowing of habits makes possible the reaping of character. The repeating of proper acts requires thought in the selection of the acts if good habits are to result. The absence of thought results in bad habits because it is easier to do careless acts. The writer, therefore, would add to any young man's code:

> "Make your habits; do not let them just 'happen.'"

Every accepted professional code of practice agrees with the pronouncement that self-laudatory advertisement is dishonorable. With this the writer would include,

> "Self-overvaluation is offensive and will always be resented and over-discounted."

One of the first dilemmas that comes to a young college graduate often involves the ethics of securing employment and of accepting a job after he has had several applications to different firms favorably received. It is not always possible for the student's first job to be in the exact field of endeavor that he has thought he would enter. In most cases, no doubt, he will accept the first employment he may be offered in his general field of endeavor. In accepting this first employment, the student often does so with the mental reservation: "I'll quit this job the minute I find something better to my liking," and all too often he does just that, with never a thought as to the ethics of his action. Perhaps this employer has given the young man a job that was to be of considerable duration and his first months of work constitute a "breaking in" or training period; as such, it is a liability to an employer. It is likely that the employer considered several applicants, some of whom were sincere in there desire to secure the job. Under such conditions the young engineer quits

after a short period of service, and the employer is materially damaged and suffers valuable time lost. Of course, the employer takes care to prevent a recurrence of the episode by one of two precautions. He either becomes suspicious of all young college graduates or decides that henceforth he will waste no time on graduates of this or that engineering school. The writer is of the opinion also that such an employer may question the value of the services of some engineering educator, perhaps justly so.

The entire question of proper conduct in seeking, securing, and holding employment by young college graduates is worthy of having a more complete treatment and, although such conduct is a matter that most any graduate should be capable of handling ethically, unfortunately it is one of the first to be violated.

A young college senior wants a job to follow commencement; he writes numerous applications and interviews personnel directors. After a weary time of waiting, during which he receives several replies stating that there are no openings, he discovers an offer of a job in his mail and in high spirits he accepts hurriedly, even if commencement is still six weeks or more away. Now that he is assured of a job, his cares are over until several weeks later when he receives another offer of a job at $25 more per month. Of course a young engineer asks himself the question: "Now, what shall I do?" He needs the best job and "best" to him at the moment may and often is the one that will help him the most in paying his college debts. Shall he accept the best paying job and write a letter to retract his first acceptance, or just what can he do? The old practitioner knows the answer; probably the student also knows, but it is difficult for him to realize that a code of proper conduct may appear to penalize him. As a matter of fact, the faithful adherence to a worth-while code does perhaps "cost" something in a sense, but when the subject is analyzed, one invariably comes to the conclusion that in the end the "price" is cheap indeed and the paying of it gives one his self-respect.

The writer commends the author for including one section devoted to the ethics of business relations. The practice of so-called sales-engineering, wherein the self-styled or trained engineer becomes a salesman in the distribution of construction materials, furnishes a field of endeavor for many men. Is the change of status from engineer to salesman justification for an individual to discard an engineer's code and set up a rule which substantially says: "Success is to be had only as sales increase, regardless of how the increase may be accomplished"? Obviously, honesty can have only one meaning, and misrepresentation in sales practices is precisely as dishonest and unethical as false design, careless work, or other indifferent engineering practice.

Good reputation is something that is cherished by every human; and good reputation results not only from a life well lived, but also from experience gained in work well and properly done. Proper ethics adds to reputation, and long adherence to proper standards of professional conduct increases reputation immeasurably. One may not be the right kind of engineer today, the salesman tomorrow with a different code, and a reputable engineer again next week. Good reputation, therefore, is the result of good habits and requires time, diligence, and patience to establish. Gained otherwise it may not and probably

will not endure. Josh Billings stated[11] it correctly, although perhaps inelegantly, with the remark: "Lasting reputashuns are a slow growth; the man who wakes up famus sum morning, iz apt to go to bed sum night and sleep it all off."

W. L. WATERS,[12] M. AM. SOC. C. E. (by letter).[12a]—Professor Mead's comprehensive paper can be divided into two parts: (1) That relating to the technical education and development of the young engineer, which is the usual advice given to college students; and (2) that relating to personal ethics and conduct. Part 2 is somewhat long and detailed so that, perhaps, it loses some of its "punch." It should be sufficient to say that "every engineer should at all times think and act in accord with the highest principles of personal honor." If anything more elaborate is desired, nothing could be better than Thomas Van Alstyne's code of honor which is given at the end of the paper (28). It is truly a classic, and all must feel indebted to Professor Mead for spreading it on the Society records.

Some may consider that in this most excellent paper (which it is stated is principally for the younger engineers) there is a slight touch of the "holier than thou" attitude of the older men. It is youth which must preserve the high ideals and standards of honor. By middle age most men have become somewhat materialistic; and if they are honest with themselves, they look back with some longing to the high ideals of their youth. In thirty-five years of experience the writer has never once known an engineer less than thirty years of age to act professionally in any other way than in accord with the highest standards of honor. Would that he could say the same of those older than forty.

The young engineer is told that he must not compete for work on the basis of "cut-rate" fees. One of the most highly respected consulting engineers abroad "broke into" a certain line of work by cutting his fees below the accepted standard percentage, and was criticized for so doing at the time; but now all that is forgotten and he stands at the top of his profession. Others have done the same. So the temptation for young engineers to do likewise is evident.

Corrections for *Transactions:* In Section IV, Paragraph 6, line 1, after "prepared," insert "especially for him"; in Section V, Paragraph 5, line 2, after "failure" insert "of purpose"; and in Section VII, Paragraph 6, line 2, after "responsible" insert "qualified."

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[11] Josh Billings' Farmers' Allminax.

[12] Cons. Engr. (Bury & Waters), New York, N. Y.

[12a] Received by the Secretary February 3, 1940.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

DISCUSSIONS

RELATION OF THE STATISTICAL THEORY OF TURBULENCE TO HYDRAULICS

Discussion

BY BORIS A. BAKHMETEFF, M. AM. SOC. C. E.

BORIS A. BAKHMETEFF,[30] M. AM. SOC. C. E. (by letter).[30a]—There scarcely can be a subject of greater importance to the hydraulic engineer than the phenomenon of turbulence. Its outward apparent aspects have been known for years. Systematic detailed studies, however, and particularly quantitative appraisal of the inner mechanism, are a matter of recent development. So far, most of the investigations were connected with the aeronautical field. Mr. Kalinske is "breaking ground" in the domain of hydraulics. No praise is too high for this worthy initiative.

In the study of turbulence, as in other domains of physical science, the first requirement is the acquisition of facts—dependable records of actual observation. Observational technique in the domain of turbulence is exceptionally difficult. For air, the present-day needs are satisfied by the hot-wire anemometer, particularly as perfected by the National Bureau of Standards. For liquids, the hot-wire method has not asserted its usefulness; so recourse must be taken to photographing molar movements, rendered visible by that, or other, optical means. The outstanding achievement in the past is the work of A. Fage and H. C. H. Townend. Mr. Kalinske has come forward with new suggestions in this line. The successful development and application of the new technique constitutes, in the view of the writer, the principal contribution of Mr. Kalinske's paper. The experimental curves, as shown, justify the hope that the complex features, specifically characteristic of turbulence as it occurs in hydraulics, may some day really become revealed and elucidated. For the moment one may only regret that Mr. Kalinske did not describe his technique in greater detail.

A drawback of all cinematographic methods is their natural tediousness. For the moment that is an inescapable evil. No greater service could be

NOTE.—This paper by A. A. Kalinske, Esq., was published in October, 1939, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: January, 1940, by Hunter Rouse, Assoc. M. Am. Soc. C. E.; and February, 1940, by Messrs. Martin A. Mason, and J. C. Stevens.

[30] Prof., Civ. Eng., Columbia Univ., New York, N. Y.

[30a] Received by the Secretary February 3, 1940.

rendered to the hydraulic profession, than if some ingenious inventor conceived a device that would permit observing and registering turbulence in liquids directly. The problem stands as a challenge, open to competitive imagination.

Mr. Kalinske purports to recapitulate briefly the principal results attained by the statistical method. The achievements of the English school are particularly emphasized. Perhaps the author has tried to encompass too much material into too narrow a space, because the nearly laconic brevity might be found somewhat trying. The paper, on the other hand, should rightly be considered as a pioneer attempt, and the engineer should not feel discouraged by the first encounter with a subject which, in the course of years, is certain to become a familiar every-day tool for whoever will be striving to penetrate and understand the baffling mysteries of the behavior of "water."

So far, the most tangible contributions of the statistical approach are not in the realm of a general all embracing "theory." With all the brilliance in recent advances, the "theory" has not reached the state of lucid certainty, which could make it a useful tool in the hands of the engineer. Statistical treatment, on the other hand, has matured into certain well-established basic definitions, which permit one to appraise and characterize turbulent phenomena in plausible numerical terms. The curves in Mr. Kalinske's paper pertaining to such definitions would seem to indicate the way through which these new concepts can be best conveyed to the "mind" of the profession. No tool is really available until it becomes physically tangible and until its dynamical essence can be explained in the simplest terms. The surest path is to lay open physical facts and make clear their practical meaning and significance. This is the next step to be achieved by the interpreters of the theory.

A question of especial interest is the spatial distribution of turbulent intensity in a cross section. The matter bears directly on efforts to unveil the mystery of the origin and spreading of turbulence. The writer is particularly interested in the form of the $\bar{u}$-curve as given in Fig. 7. As mentioned before, factual knowledge of turbulent phenomena is more than scarce and no opinions of too categorical a nature are warranted at this stage. Nevertheless it does seem that all observations heretofore concur,[31] that the axial component continually increases from the center toward the wall, at least to within the unstable border zone next to the laminar sublayer. The Fage and Townend[32] ultramicroscopic photos show the largest fluctuations as being in the closest proximity (distance less than 0.003 in.) to the wall. More recently such conceptions have been substantiated by observations at the Massachusetts Institute of Technology.[33] They stand, moreover, in close relation to the present-day views on the pendulatory character of the motion in the laminar zone, so admirably elucidated by Dryden.[34]

The writer would like to learn from Mr. Kalinske whether the particular outline of the $\bar{u}$-curve, shown in Fig. 7 (which reaches a maximum at about one

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[31] "Statistical Measurements of Turbulence in the Flow of Air Through a Pipe," by H. C. H. Townend, *Proceedings*, Royal Soc. of London, Vol. 145A, 1934, p. 180.

[32] "An Examination of Turbulent Flow with an Ultramicroscope," by A. Fage and H. C. H. Townend, *loc. cit.*, Vol. 135A, 1932, p. 656.

[33] Discussion by J. C. Hunsaker of the Second Wright Brothers Lecture, *Journal of the Aeronautical Sciences*, Vol. 6, 1939, p. 105.

[34] "Turbulence and the Boundary Layer," by Hugh L. Dryden, *loc. cit.*, Vol. 6, 1939, p. 85.

half the distance and then declines) has been generally substantiated by repeated observations or may be ascribed to some accidental reason. The point is of vital interest in its possible bearing on the gradually crystallizing conception, that the eddies, which make for turbulence, are principally generated in a special active zone growing out of the instability of the laminar sublayer. Prandtl[35] most aptly characterized this zone as a "vortex mill" (Wirbelfabrik), assuming that the central portions of the fluid, into which the eddies are "cast off," play a more or less passive role.

The foregoing remarks tend to show how uncertain, as yet, is current knowledge of the inner mechanism of turbulence. They do not detract in any way from the value of Mr. Kalinske's contribution.

[35] *Proceedings*, Fifth International Cong. for Applied Mechanics, 1938, p. 367.

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DISCUSSIONS

NORRIS DAM CONSTRUCTION CABLEWAYS

Discussion

BY J. S. FOSTER, ESQ.

J. S. FOSTER,[3] ESQ. (by letter).[3a]—Facts and computations relating to the overhead cableways used in the construction of the Norris Dam are presented concisely and interestingly in this paper, and the authors are to be congratulated for their careful and comprehensive work.

The computations for cable and rope stresses which, as the authors state, are according to accepted formulas probably have never previously been checked with the actual stresses in similar installations—certainly not in such a thorough manner. The writer believes that there is not available a tensometer registering to nearly 400,000 lb as required on this installation if applied directly to the main cable; therefore, the use of the smaller tensometer on each of the 10 "parts" of the take-up as was done by the authors should give a substantially accurate result. It is apparent, as shown by the curves in Fig. 11, that stresses at point (1) (about 32% of the span) would be somewhat less than calculated or actual results if taken at midspan, about 960 ft from the head tower.

The authors' explanation of moderate impact in lifting the load at the point of pickup is logical and is to be expected; the flexible main cable "droops," with only gradually increasing resistance to the pull of the hoist rope, until deflection of the main cable under the load at the pickup point ceases—without doubt minimizing impact as the load leaves the ground.

In comparing computed stresses plus 25% impact as shown in curve *B*, Fig. 11, with tests including impact as in curve *E*, Fig. 11, it is a question of which result (calculations or test by tensometer) is more accurate; however, it is an interesting showing, and tensometer test with the loaded carriage at rest at different points (for instance at 1, 2, 3, 4, 5, 6, and 7, or a few of them) compared with calculated stress without impact would be interesting, because it is not quite clear from Fig. 11 that such results have been obtained. The al-

NOTE.—This paper by R. T. Colburn, M. Am. Soc. C. E., and L. A. Schmidt, Jr., Assoc. M. Am. Soc. C. E., was published in December, 1939, *Proceedings*. This discussion is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion of the paper.

[3] Lidgerwood Mfg. Co., Elizabeth, N. J.

[3a] Received by the Secretary February 13, 1940.

lowance made in design calculations for an impact of 25% of the live load could have been assumed only.

The cableway towers embody the Ackerman type of thrust-wheel, as described and illustrated in the paper, and their thorough and rugged construction is well shown in Fig. 6, as compared with traveling towers with inclined front track having inclined wheels and horizontal track with vertical wheels at the rear, all with single flanges. The latter type of towers was used on the cableways for Conchas Dam and other dams in the West. In calculating the horsepower required to move the traveling cableway towers with full load suspended, it is essential to consider the power necessary to accelerate not only the tower with its machinery and counterweight, but also the cable, ropes, carriage, bucket, and load from rest; but moderate speed and moderate accelerating time only are required, so that, with level track and reasonably well-lubricated track-wheel journals, required power to accelerate may be controlled. Possible misalinement of the two towers, or slight irregularity of the trackway as suggested by the authors, may account for the seemingly heavy pull required to move the towers at Norris Dam. The radial cableway towers at Conchas Dam were moved by traction, and misalinement was unlikely. These Conchas Dam towers (145 ft high, each weighing about 300 tons, including the machinery and counterweight) required only about 11 hp to move one tower at a uniform speed of about 75 ft per min; to accelerate required about 98 hp.

In view of the cost of renewing the main cables, it is important that they be operated at stresses and carriage-wheel loads within proper limits; and it is obvious that they were so operated on the Norris Dam. Each main cable finished the job, handling more than 1,000,000 tons, and at least one of the original main cables was used afterward on another dam being constructed by the TVA.

The data on effect of temperature change on the length of the main cable are interesting and had not previously been published in connection with cableway work.

The tensometer tests on operating ropes shown in Table 8 do not show much variation from calculated stresses except for the outhaul rope. Variation in this rope is due to the fact that it is set up with a considerable initial tension to give necessary adhesion for driving and to avoid too much slack on the inhaul side of the rope. The conveying drum in this case is spool-shaped and the endless rope surges back and forth. This general practice is necessary on long spans to avoid the expense of very large single drums or an additional rewinding drum. Shorter spans use separate ropes with fixed ends on one drum for inhauling and outhauling. The considerable stress is also due partly to the fact that the maximum stress given occurs when the load is being hauled up the steep grade approaching the tail tower.

Apparently, the type of bucket used on Norris Dam was very successful. As a matter of fact the aerial or automatic dump bucket, using a third drum enabling the cableway leverman to open, discharge, and close the bucket from his station in the head tower, was used throughout on Hoover Dam, Parker Dam, and other important dams. This third drum, automatic dump also will be used on the cableways for building Shasta Dam.

The data on output and cost of operation are very interesting and indicate that two cableways placed concrete in one shift at the rate of 180.7 cu yd per hr which was the limit of mixer output; one cableway actually placed 162 cu yd in 1 hr, but normally worked at a continuous rate of 120 cu yd per hr.

Operating cost per ton, exclusive of depreciation, appears to be about $3\frac{1}{2}$ cents, or equivalent to about 7 cents per cu yd on the basis of $11.44 per hr as given at the end of the paper.

The cost of electric current given under the heading "Operation and Maintenance: Performance"—namely 66.2 kw per hr, for each cableway—is moderate and somewhat less than on contract work on a notable installation on the west coast.

Actually the salvage value realized would probably be considerably more than $100,000 in view of the fact that the paper does not include salvage of certain material apparently unsold at the time data were compiled. On contract work, where the experienced contractor has the opportunity of continuing the use of the cableway on other contracts, no such depreciation need be considered. Obviously, the contractor who buys secondhand machinery, if it is in good condition, leaves most of the depreciation with the original purchaser; but this advantage of the secondhand buyer may easily be lost in the difference in cost of operation between out-of-date or damaged machinery and new machinery of more modern design and construction.